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MODEL STUDY OF A DYNAMICALLY LATERALLY LOADED PILE[a]

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(Proc. Paper 1535)

ABSTRACT

A dimensionally scaled model of a vertical pile in soft soil has been dynamically tested. Results indicate that a low frequency oscillatory lateral load induces pile bending moments which closely correspond to moments caused by the same load applied statically. Analytical computation of pile moments based on the assumption of a soil modulus constant with depth appears to agree well with dynamic test results.

SYNOPSIS

This study deals with the analytical and experimental analysis of a laterally loaded vertical pile embedded in a weak marine foundation. Special emphasis in the experimental phase is placed upon non-impact dynamical loading of the freehead pile. In particular, the applied lateral load oscillates sinusoidally, similar to that produced by ocean waves.

The method employed herein of obtaining experimental results is based upon the principles of dimensional analysis and dynamical similarity. A laboratory model pile was designed by these principles and electronically instrumented to simultaneously record applied loading and induced stresses at selected locations along the model pile. The study is of interest from two standpoints; it uses a dynamically scaled model and the instrumentation allows

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simultaneous evaluation of applied force and resulting bending moments induced along the pile under dynamic conditions.

Test results indicate that there is practically no difference in bending moments caused by a dynamic lateral load as compared to a static lateral load. The pile vibrates in a standing wave form which is at all depths in phase with the applied load. Pile bending moments computed by the analytical method assuming a constant soil modulus were in good agreement with dynamic test results obtained from the model.

Notation

a	Acceleration
β	Constant dependent upon pile modulus of elasticity, pile moment of inertia, and soil modulus
c	Distance from neutral axis to outer surface of pile
C_M	Dimensionless inertial coefficient
d	Linear distance
D	Diameter
Δ_c	Amplitude of run calibration trace
Δ_b	Amplitude of test run race
E	Modulus of elasticity
E_s	Modulus of elasticity of soil
f	Maximum bending stress at a point along a pile
$f(x)$	Function of the variable x
F	Force
g	Acceleration of gravity
I	Area moment of inertia
k	A constant of proportionality
K_D	Dimensional drag coefficient of viscous fluid flow
L	Length
m	Subscript denoting model
M	Mass or mass per unit length when applicable to model similitude Bending moment when applicable to pile bending
M'	Virtual mass of displaced soil
M_e	Effective mass (sum of M and M')
n	Model scale factor
p	Subscript denoting prototype
P	Concentrated load
π	Dimensionless and independent quantity

r	A constant
ρ	Density
T	Time
ω	Frequency of oscillation
x	Depth below mud line
X^{IV}	Fourth derivative of X with respect to x
y	Lateral pile deflection
y_o	Amplitude of a simple harmonic oscillation

INTRODUCTION

As coastal development and the oil industry began to extend their activities offshore, a problem was encountered whose nature was both engineering and oceanographic; the design of laterally loaded piles. Many coastal areas are characterized by poor foundation conditions so a knowledge of the fields of soil mechanics and marine geology became important to their exploitation. Obviously, any study of pile strengths in such environments must recognize the interaction of wave, pile, and foundation.

When it became necessary to build an offshore island, such as an oil drilling platform, the emphasis on piling design criteria shifted from the static vertical loads imposed by the structure to the dynamic lateral forces exerted by ocean waves on the structure. In an area such as the Gulf Coast the problem was further complicated by the widespread occurrence of soft, poorly consolidated clay foundations. Attempts to predict pile deflections and moment distributions analytically in these low strength foundations have been based on earlier engineering studies of pavement and railway track deflection. Even the simplest experimental studies on large piles or groups of piles have been limited in number due to the high cost of special equipment and technical manpower.

The primary objective of this study is to investigate the dynamic effects of simulated ocean wave action on a single model pile embedded in a soft simulated marine foundation and to consider these results in light of a selected prototype. Both analytical and experimental methods are used to obtain the comparison of pile moments induced by dynamic loading with those obtained under static conditions.

The Laterally Loaded Pile

The problem of a vertical pile embedded to a depth of the order of 100 feet or more in a soft marine sediment is somewhat similar to the classical problem of an infinitely long beam on an elastic foundation (Winkler, 1867; Timoshenko, 1941). The essential differences in these problems are: (1) the finite length of the pile (importance decreases as the slenderness ratio becomes very large), (2) the relatively inelastic condition of a natural sediment as the medium resisting pile flexure, and (3) the lack of homogeneity of the soil in respect to both elastic modulus and maximum strength. The latter two items are perhaps the most important and have been the subject of some

further study which will be discussed below.

A foundation medium may generally be thought of as being horizontally stratified or, in the case of a marine sediment, becoming more consolidated with depth. Consequently a horizontal beam placed at any depth would tend to experience a uniform soil pressure over a given length. Hence, assuming a perfectly elastic foundation, there developed the concept of a constant soil modulus of elasticity which, when multiplied by the beam deflection at a point, would give the resisting pressure exerted by the foundation. However, in the case of a vertical pile, it seems obvious that the soil modulus will usually increase with depth. In order to employ the method described by Winkler and Timoshenko, it is necessary to replace this variable soil modulus with an equivalent constant modulus which may be used for the pile length under consideration. The deflection, y, of a long vertical pile in a medium of uniform soil modulus, E_s, is obtained from the general equation (Chang, 1937):

$$y = e^{-\beta x} \left(C \cos \beta x + D \sin \beta x \right) \tag{II-1}$$

where $\beta = \sqrt[4]{\frac{E_s}{4EI}}$, x is depth below mud line, E is pile modulus of elasticity, I is pile area moment of inertia, and the constants C and D depend upon fixity at the head of the pile. Since the parameter, β, is a function of the fourth root of the soil modulus, engineers have generally rationalized that it is unnecessary to determine the modulus accurately because a large variation in it will produce only a small difference in β.

Theoretical analysis of a pile subjected to a static lateral load in a soil which is assumed to have a soil modulus that varies with depth has been most successfully approached by means of a difference equation solution. The basic differential equation governing the situation is

$$EI\frac{d^4y}{dx^4} + E_s y = 0 \tag{II-2}$$

where $E_s = f(x)$ and is the assumed soil modulus. A complete solution of Eq. (II-2) by finite differences was first presented by Gleser (ASTM, 1953) and later revised by Focht and McClelland (1955) in an attempt to reduce the tedious and time consuming task of working out a complete deflection curve. This method has the distinct advantage of placing no restrictions on the soil modulus variation.

A large number of experimental tests have been performed on laterally loaded piles and piling systems embracing a great range in length, size, and material of the pile, condition of the foundation, and fixity of the pile head (ASTM, 1953; Reeves, 1955; Matlock and Ripperger, 1956; Texas A & M Research Foundation, 1952). Most of the tests were conducted to aid in a specific design, and experimental equipment varied from miniature models to full scale prototypes. These data have formed the foundation upon which present-day design criteria are based. This study treats the laterally loaded pile in a very soft foundation medium so the value of soil modulus, E_s, is intended to represent a lower limit of moduli which might be encountered under natural conditions.

One characteristic has been common to all the experimental testing conducted to date; namely, static loading at the time of stress, strain or moment measurements. The most common test apparatus employs some device for

measuring strains at fixed locations on the pile. Strains are converted to stresses and the distribution of bending moment in the pile as a function of depth is then determined. By integrating twice and applying appropriate measured or inferred end conditions, the deflection curve is obtained and by differentiating twice, the soil pressure curve may be evaluated (Fig. 1). Obviously the accuracy of each of these curves is dependent upon the accuracy maintained throughout the procedure. However, in the final analysis, the results still pertain to static loading so that accurate or not, they are at best only an approximation of what might actually happen under dynamic conditions.

The very recent series of test undertaken at Lake Austin (Matlock and Ripperger, 1956) was the first attempt to analyze the effects of dynamic loading. It appears to the writer that these tests fall in a special category lying between static and dynamic conditions because the measurements were made after the pile had been dynamically loaded and while the head was held statically with a fixed deflection. It may also be noted that although the instrumentation was very precise, the time required to take a complete series of measurements was considerable. This indicates that if measurements were begun immediately after the oscillatory loading was stopped both the soil and pile would be in a state of readjustment during the measuring period. Results might vary considerably depending upon when the measurements were begun and the length of time taken to make a complete set of measurements.

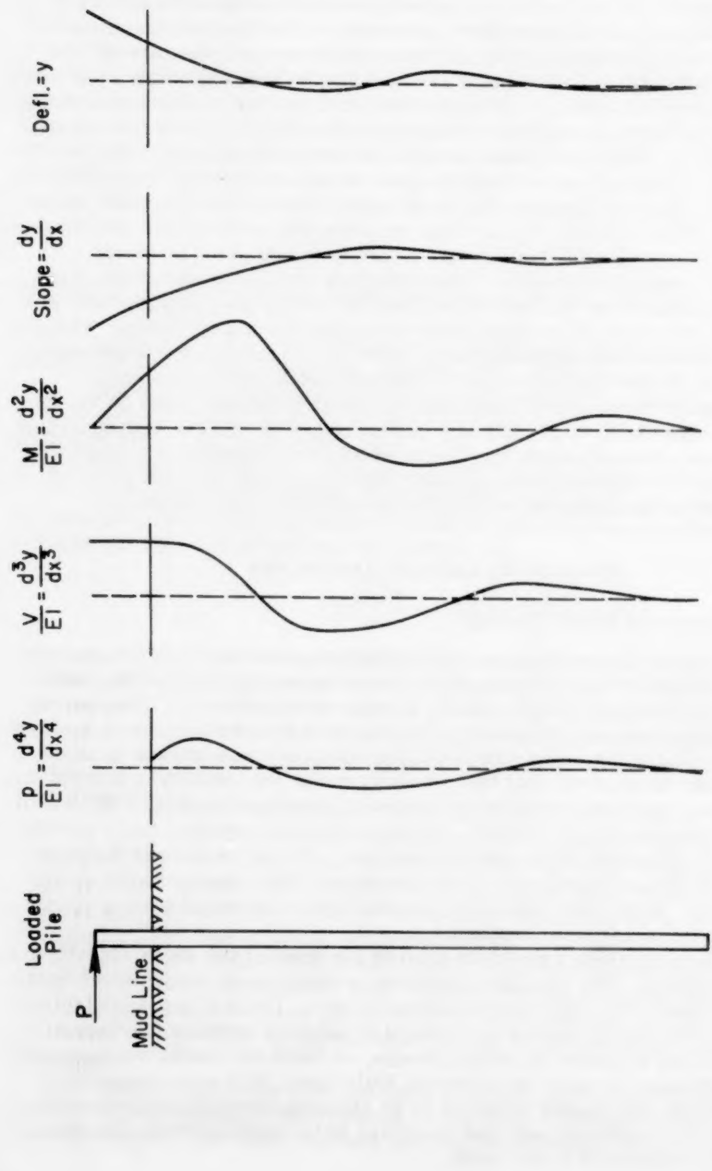
To the best of the author's knowledge, the tests to be described on the following pages are the first of their kind in two respects: (1) the test apparatus is a dynamically scaled model, the design of which is based upon a "typical" prototype, and (2) a dynamic lateral load, simulating that of ocean wave action, is applied and strains induced in the pile are simultaneously measured.

Model of the Laterally Loaded Pile

Practical Aspects of Model Testing

To date, testing of prototype or semi-prototype piles has been the primary approach to experimental analysis of the laterally loaded pile (ASTM, 1953). Economically, prototype testing usually is not overly desirable. The cost of materials, equipment and manpower often becomes prohibitive if the tests are to produce valid results. Time required for construction, instrumentation, and testing may be so great that test results greatly lag industry's demand for them. Perhaps the most undesirable feature of prototype testing is its lack of versatility. Results are, in general, highly specialized and applicable only to the particular apparatus in its own environment. It may be difficult to make alterations on such a large piece of equipment and also impractical to change the testing site. In the final analysis, results from a series of tests apply to a particular pile in a special location.

Model testing provides a possible remedy for most of the above objections to prototype testing. The smaller apparatus is much more inexpensively and conveniently fabricated with a great saving in time. Little if any work is required in the field and no delays are caused by adverse weather conditions. Special equipment is easier to obtain, design, or build and modifications to the original design may be made at relatively little cost. Still more important, the apparatus may be readily modified to fit changing demands or to investigate the effects of changing selected variables while holding others constant. Control is the essence of model study.



CHARACTERISTICS OF A STATICALLY LATERAL LOADED FREEHEAD PILE

FIGURE I

Model Design

Principles governing dimensional analysis as applied in model design have been summarized by various investigators (Buckingham, 1915; Gibson, 1924; Murphy, 1950). Design of a model and interpretation of its results are founded on a special application of dimensional analysis known as the Buckingham Pi Theorem. This theorem states that "the number of dimensionless and independent quantities required to express a relationship among the variables in any phenomenon is equal to the number of quantities involved, minus the number of dimensions in which those quantities may be measured". This means that the development of specifications for a particular model is essentially a two-step process: (1) establish all of the independent factors that significantly influence the behaviour of the physical system to be studied and (2) combine these using the principles of dimensional analysis to obtain a functional relationship. The exact nature or arrangement of terms within each nondimensional group is arbitrary; the only requirement is that each group be independent.

Model Pile Design

The following nine variables, considering EI as a single variable, were chosen for the basis of design:

Variable	Dimensional Units	Description
x	L	Vertical distance from point of load application to any point on the pile
y	L	Lateral deflection at x
D	L	Pile diameter
P	MLT^{-2}	Lateral applied load
E	$ML^{-1}T^{-2}$	Modulus of elasticity of pile
I	L^4	Moment of inertia of pile cross-section
E_S	$ML^{-1}T^{-2}$	Modulus of elasticity of soil
M	ML^{-1}	Mass per unit length of pile
ω	T^{-1}	Frequency of load oscillation
ρ	ML^{-3}	Density of mud

The symbols L, M, and T are here used to represent length, mass and time respectively. Admittedly such parameters as viscosity of the soil and variations of E_s with x are neglected.

Since a total of three different units and nine variables are used, six "π" terms (Buckingham, 1915) are required to relate all the model parameters to those of the prototype. The π term y/D was related to five other π terms:

$$\frac{y}{D} = f\left[\frac{x}{D}\ ;\ \frac{EI}{PD^2}\ ;\ \frac{EI}{E_s D^4}\ ;\ \frac{M}{\rho D^2}\ ;\ \frac{P}{M\omega^2 D^2}\right] \qquad \text{(III-1)}$$

It should here be noted that the third π term on the right hand side of Eq. (III-1) is merely $\left(\frac{4}{\beta D}\right)^4$. Also, the fourth π term is practically equivalent to the pile inertial coefficient, C_M, or the ratio of the mass of the pile to the mass of the soil it displaces.

Since all of the π terms are a function of the diameter, D, to some power r, by letting $n = \frac{D_p}{D_m}$, each of the π terms relates model to prototype by the parameter n^r. That is:

$$\left(\frac{y}{D}\right)_p = \left(\frac{y}{D}\right)_m \text{ or } y_p = nx_m \qquad (1)$$

$$\left(\frac{x}{D}\right)_p = \left(\frac{x}{D}\right)_m \text{ or } x_p = nx_m \qquad (2)$$

$$\left(\frac{EI}{PD^2}\right)_p = \left(\frac{EI}{PD^2}\right)_m \text{ or } \left(\frac{EI}{P}\right)_p = n^2\left(\frac{EI}{P}\right)_m \qquad (3)$$

$$\left(\frac{EI}{E_s D^4}\right)_p = \left(\frac{EI}{E_s D^4}\right)_m \text{ or } \left(\frac{EI}{E_s}\right)_p = n^4\left(\frac{EI}{E_s}\right)_m \qquad (4)$$

$$\left(\frac{M}{\rho D^2}\right)_p = \left(\frac{M}{\rho D^2}\right)_m \text{ or } \left(\frac{M}{\rho}\right)_p = n^2\left(\frac{M}{\rho}\right)_m \qquad (5)$$

$$\left(\frac{P}{M\omega^2 D^2}\right)_p = \left(\frac{P}{M\omega^2 D^2}\right)_m \text{ or } \left(\frac{P}{M\omega^2}\right)_p = n^2\left(\frac{P}{M\omega^2}\right)_m \qquad (6)$$

where the subscript "p" applies to the prototype and "m" applies to the model.

To design the model for this study, a "trial" prototype pile was chosen with the following approximate specifications: D = 136 in., wall thickness = 1 in., $E = 30 \times 10^6$ psi, $\rho g = 110$ pcf, $E_s = 100$ psi. After several trial designs, a final model pile was chosen with specifications as follows:

Aluminum pipe (pile)	3003 - H112
Outside diameter, D_m	= 2.375 in
Inside diameter	= 2.067 in
Wall thickness	= 0.154 in
Unit weight, $M_m g$	= 1.264 lb/ft
Length of pile, L_m	= 96 in
Moment of inertia, I_m	= 0.666 in^4
Modulus of elasticity, E_m	= 10.3×10^6 psi

Soil specific weight $(\rho g)_m$	= 70 pcf
Frequency of cyclic load, ω_m	= 6.28 rad/sec
Period of cyclic load	= 1.0 sec
Assumed soil modulus, E_{sm}	= 48 psi

The corresponding prototype is a 36 in. diameter steel pile in a soft clay with a specific weight of approximately 110 pcf:

Model scale factor, n	= 15.15
Outside diameter, D_p	= 36 in
Inside diameter	= 33.6 in
Wall thickness	= 1.2 in
Unit weight, $M_p g$	= 456 lb/ft
Length of pile, L_p	= 121.2 ft
Moment of inertia, I_p	= 19,884 in^4
Modulus of elasticity, E_p	= 30 x 10^6 psi
Soil specific weight, $(\rho g)_p$	= 110 pcf
Frequency of cyclic load, ω_p	= .419 rad/sec
Period of cyclic load	= 15.0 sec
Soil Modulus, E_{sp}	= 79 psi

The adoption of a constant soil modulus for the model design was based upon the assumption that the total area under the soil modulus distribution curve might provide the most satisfactory means of relating a constant modulus to one proportional to depth. John A. Focht, Jr. in a letter to the writer (November, 1955) stated values of k in the relationship $E_s = kx$ have been estimated as low as 0.5 in very soft clay deposits. Assuming k = 1 for the model under consideration, the area under the modulus distribution curves is found to be $1/2 \times 96 \times 96 = 4608$. An equivalent constant modulus is $4608 \div 96 = 48$ psi which was used for the model design.

Several features of the model pile are worthy of special note. Aluminum with its lower modulus of elasticity is used in order to allow greater lateral deflections and bending strains with lower applied loads. A convenient cyclic loading is used on the model, the prototype counterpart of which yields a realistic ocean wave period that might be used for storm wave design.

One of the most difficult problems encountered in fulfilling model design requirements was the choice of a foundation medium. From a geometric standpoint, it is virtually impossible to scale down the particle size of a representative natural marine clay. However, the geometric properties of the sediment are not likely to be important, especially when compared with its engineering properties, i.e., density, stress deformation characteristics, and shear capacity. More concisely, the validity of experimental results are dependent upon how the medium acts rather than its appearance.

Experimental Apparatus

Fig. 2(a) is a general view of the experimental test equipment. For the purpose of description, the apparatus may be considered as composed of five component parts; tank, pile, mechanical oscillator, foundation medium, and electronic equipment.

Tank Assembly

The tank (Fig. 2a) was a wooden structure with nominal horizontal dimensions of two feet by four feet and a height of eight feet. The lower three feet of the tank was set in a pit, thus allowing convenient access to the top of the tank from floor level. To prevent escape of water and consequent drying of the foundation medium, the tank was waterproofed on the inside with an asphaltum paint. An observation window was provided on the long side and at the top of the tank to observe the in place characteristics of the foundation and its possible movement during testing.

Pile Assembly

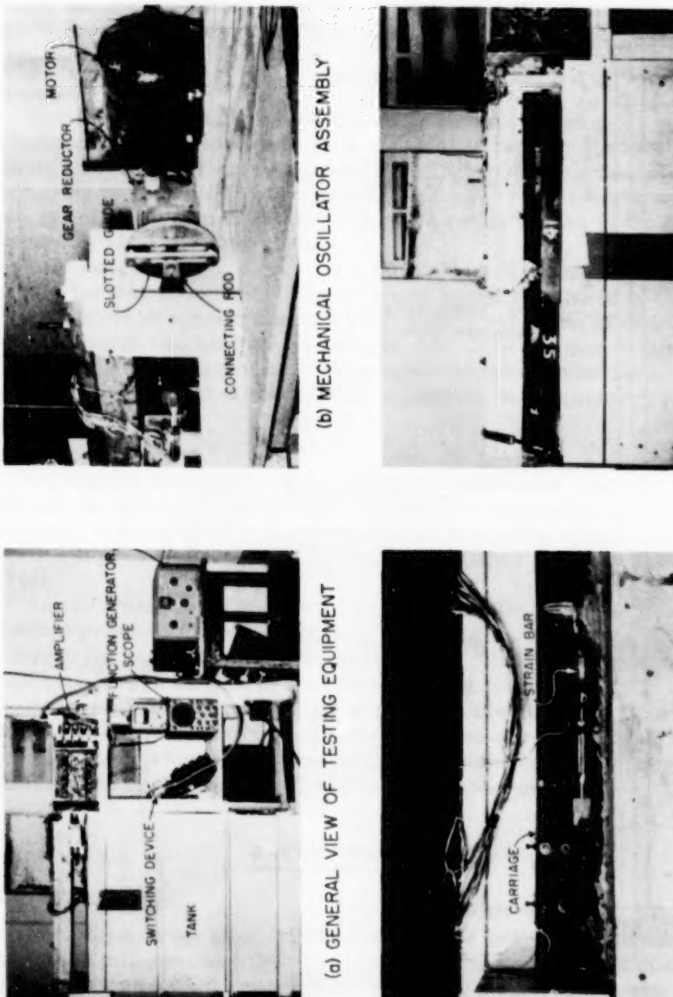
The two inch IPS eight foot long aluminum pipe was fitted with a recessed cap at the bottom end. This recess was set on a 3/8" rounded stud to provide a point support (no moment capacity) while the tank was being filled with mud and during testing. A yoke was provided 3/4" from the top of the pile with bearing holes, for the yoke cross bar, drilled on the pile's neutral axis.

Ten strain gage locations were chosen as shown in Fig. 3. Four Baldwin SR-4, type A-13, strain gages were placed in diametrically opposed pairs and aligned with respect to pile bending axis (Section A-A of Fig. 3). There was thus provided at each location a balanced electrical bridge of four gages, each gage being placed so as to measure the maximum surface strain at the pile cross section. Leads from the gages were threaded into and out of the top of the pile through 3/16" access holes drilled on either side of the pile along the neutral axis and 3/4" above each gage location. To prevent short circuits, electrical tape was used beneath the connections of gage to lead and the connections were carefully separated while water proofing was carried out in accordance with the procedure recommended by the manufacturers (Baldwin, 1951). Approximately a four inch length of the pile at each location was wrapped with electrical tape and several coats of G. E. "Glyptol" applied to the area before pile installation in the tank.

Mechanical Oscillator

Dynamic lateral loading was accomplished by means of a mechanical oscillator (Fig. 2b) which consisted of a motor, speed reducer, crank wheel, connecting rod, sliding carriage and strain bar. The 1/8 HP synchronous motor and 30 to 1 gear reductor were mounted on the equipment table fastened adjacent to the tank. As shown in Fig. 2b, a six inch disc was fitted with a slotted crank guide. Stroke adjustment was made by tightening the connecting rod stud at any desired location in the guide. The connecting rod drove a carriage equipped with eight wheels that rolled on a leveled steel bar suspended from a wooden "T" beam mounted above the tank (Fig. 2c).

The connection between carriage and pile consisted of a turnbuckle and a strain bar. The turnbuckle allowed installation of the carriage and adjustment



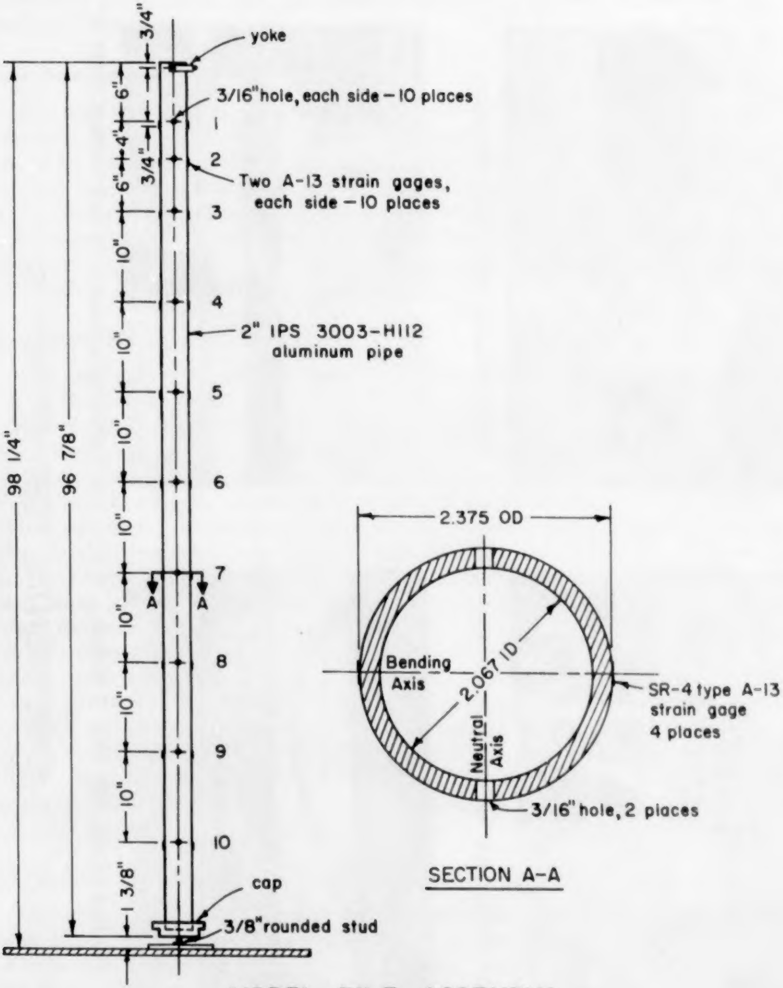
(a) GENERAL VIEW OF TESTING EQUIPMENT

(b) MECHANICAL OSCILLATOR ASSEMBLY

(c) MUD LEVEL FOR TEST 2 SERIES

(d) OVERBURDEN PLACED ON MUD SURFACE (TEST 4)

EXPERIMENTAL TEST APPARATUS
FIGURE 2



MODEL PILE ASSEMBLY
FIGURE 3

of the apparatus to pile equilibrium (no initial stress in pile) for selected stroke adjustments on the crank guide. The section of the aluminum strain bar on which two SR-4, type A-14, strain gages were mounted was .08 square inches in cross section. One end of the bar was connected to the turnbuckle and the other to the pile yoke. Before operation, the carriage, turnbuckle and strain bar were adjusted to the same elevation and leveled in order to prevent eccentricities and assure application of an axial load to the strain bar.

Foundation Medium

Montmorillonite clay, more commonly known as bentonite, was selected as a foundation or soil medium. This material is capable of absorbing a large amount of water to form a reasonably homogeneous, low density "gel" with a relatively high strength. The substance is quite elastic which would make the concept of soil modulus of elasticity more realistic than in the case of comparatively inelastic natural soils.

This montmorillonite clay has a liquid limit of approximately 600% and a plastic limit of about 50%. The clay was mixed and placed in the tank at a water content of approximately 400%. During early stages of testing a thin layer of water was kept on the exposed surface of the mud to prevent its drying out. Later a piece of oil cloth was placed over the surface. The final in-place specific weight of the foundation medium was approximately 70 pcf.

Electronic Equipment

A complete description of the electronic equipment used for this study is too lengthy to be included herein. A complete discussion by Dr. George Huebner, who designed and assembled the measuring system, is included in Section V of the full report submitted to the Office of Naval Research (Gaul, 1956).

An electronic switching device developed by Dr. Huebner allowed the simultaneous presentation of a continuous record of the highly amplified strain variations from each channel on an oscilloscope. Records were obtained by time exposure on 35 mm film of the eleven traces, corresponding to the ten gage locations on the pile and the pair of gages on the strain bar, during a single pass of the scope sweep (approximately five to seven seconds). Included in the electronic equipment and shown in Fig. 2(a) were a switching panel, amplifier, switching device, power supplies, and oscilloscope.

Procedure and Analysis

Calibration

Ten of the strain gage locations were used to measure strain due to bending in the pile and one of the locations to measure the effect of strain in the bar which was loaded axially at the top of the pile. Consequently, two separate calibrations were necessary.

Calibration of the gages along the pile was accomplished in two steps: (1) determination of the calibration factor for the gages on the pile before the pile was installed and (2) incorporation of this calibration factor into the run calibration curves (Fig. 6) put on the scope by a function generator operating at a known voltage.

Step (1) was carried out by first supporting the pile horizontally on knife edge supports placed 88 inches apart and applying a vertical concentrated load, P, of 21.15 pounds midway between the supports (Fig. 4). The bending moment M_x at any position, x, along such a simply supported beam is given by,

$$M_x = \frac{Px}{2} \; ; \; x \leq \frac{L}{2} \tag{V-1}$$

The extreme fiber stress, f_x, at position x is given by,

$$f_x = \frac{M_x c}{I} \tag{V-2}$$

where c is the distance of the extreme fibers from the neutral axis of the pile. Combining these two equations and substituting known values of P = 21.15 pounds, c = 1.19 inches, and I = 0.666 in^4, the maximum stress in psi becomes:

$$f_x = 18.88x \tag{V-3}$$

Using formula (V-3), the outer fiber stress at each gage location was determined. Measurements of the change in emf produced by applying the load were taken and a gage calibration factor then determined as shown in Fig. 4. Determinations were made only for locations 4 through 8 because the induced strain approaches zero towards the supports of a simply supported beam. All of the gages were virtually identical and the mean variation of calibration factors for the locations actually calibrated was very small, so it is felt that application of the computed calibration factor to all locations is not unreasonable.

Step (2) involved the direct comparison of the above calibration factor to a known variation or curve which could be recorded with each test run. A sine curve having a frequency of one cycle per second (same as oscillator frequency) was produced by a function generator operating at 0.53 volts rms. Through a resistance dividing network, actual applied voltage to the bridge output was .000186 volts rms. This resulted in a known uniform sinusoidal variation in all ten traces. The peak amplitude (one-half the total range) of this curve was designated Δ_c and is equal to (1.414) x (rms) or,

$$\Delta_c = .000263 \text{ volts.}$$

The calibration factor, C = 1.12 x 10^{-6} volts/psi, was then substituted into the above relationship to obtain the value of Δ_c in psi, i.e.,

$$f_c = (.000263) \left(\frac{1}{.00000112}\right) = 235 \text{ psi} \tag{V-4}$$

Again known values of c and I may be substituted in the equation

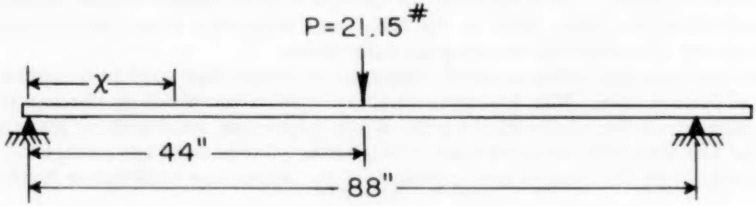
$$M = \frac{fc}{I} \tag{V-5}$$

to obtain

$$M = 0.56 \, f \tag{V-6}$$

Substituting f_c = 235 psi into Eq. (V-6), the bending moment in inch-pounds equivalent to Δ_c is found to be;

$$M_c = 131.6 \text{ inch-pounds} \tag{V-7}$$



Gage Loc.	X	100E (P=0)	100E (P=21.15)	$\Delta E \times 10^6$	Stress, f (psi)	$\Delta E/_f \times 10^6$ (volts/psi)
4	26	.4190	.4745	555	490.7	1.131
5	36	.6980	.6230	750	678.9	1.105
6	46	.0898	.0030	868	793.1	1.095
7	56	.3510	.2820	690	604.9	1.141
8	66	.1140	.0665	475	414.5	1.146

Average $\Delta E/_f \times 10^6 = 1.124$

Calibration Factor $= 1.12 \times 10^{-6}$ volts/psi

TEST PILE CALIBRATION

FIGURE 4

It should be noted that the data traces are obtained during dynamic loading and their calibration is based upon the change in gage output induced by static loading of the test pile. That is, the difference between a static and dynamic force is not accounted for in the gage calibration.

Several attempts were made to calibrate the strain bar used to measure applied lateral load. Fig. 5 summarizes the calibration which is similar to that carried out for gages on the pile. Gage output was read with no load on the bar and then with an axial load of 50 pounds. Division of the change in voltage (ΔE) by the applied load resulted in the strain bar calibration factor:

$$C = 14 \times 10^{-6} \text{ volts/lb} \tag{V-8}$$

Because 500 ohm gages were used on the strain bar, actual applied voltage to the bridge with a 0.53 rms input was .000265 volts rms. Peak amplitude (Δ_c) equals (1.414) x (.000265) or .000375 volts. Therefore from Eq. (V-8),

$$\Delta_c = 26.8 \text{ pounds} \tag{V-9}$$

As in the case of the pile calibration, the strain bar dynamic calibration is based on a calibration factor computed from static loading. It is also evident from Fig. 5. that separate calibrations gave a considerable variation in computed calibration factor.

Data Collection

Both static and dynamic tests were conducted on the model. The nature of the equipment made it necessary to use entirely different procedures for each type of testing.

The rate of oscillation (lateral wave loading) for dynamic testing was closely controlled at a frequency of one cycle per second by the 1800 rpm synchronous motor and 30 to 1 gear reductor. The magnitude of lateral load, which corresponds in a non-linear manner to the height of the 15 second prototype ocean wave, was controlled by adjusting the stroke of the connecting rod. After the stroke had been set for a given run, the guide slot was rotated to a vertical position and the pile adjusted to equilibrium by means of the turnbuckle between carriage and strain bar. A metal pointer had been placed on the tank overhead "T" beam and was matched with a mark made on the pile before the oscillator linkage had been connected. This assured that, within a reasonable degree of accuracy, the pile would encounter the same soil resistance in both directions of movement from its median position.

Variations in gage output voltage, which is directly proportional to strain at the gage location, were presented continuously on an oscilloscope by means of the electronic equipment previously mentioned. A 35 mm oscilloscope camera was used to record the simultaneous traces of all eleven gage locations during dynamic loading. The scope sweep speed was held to approximately one pass per seven seconds which provided a maximum of seven complete wave lengths for one time exposed picture. Fig. 6 presents representative examples of the dynamic test data analyzed for this study.

The traces recorded for each run in Fig. 6 are identified as follows. The uppermost trace is a straight line and is merely a reference line. From top to bottom, the ten traces beneath the reference line correspond to the ten gage locations on the pile as shown in Fig. 3. The bottom trace represents the strain bar variation (lateral applied load). Beneath each photograph of test

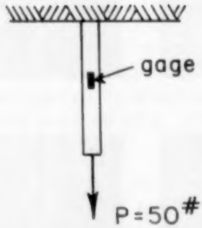


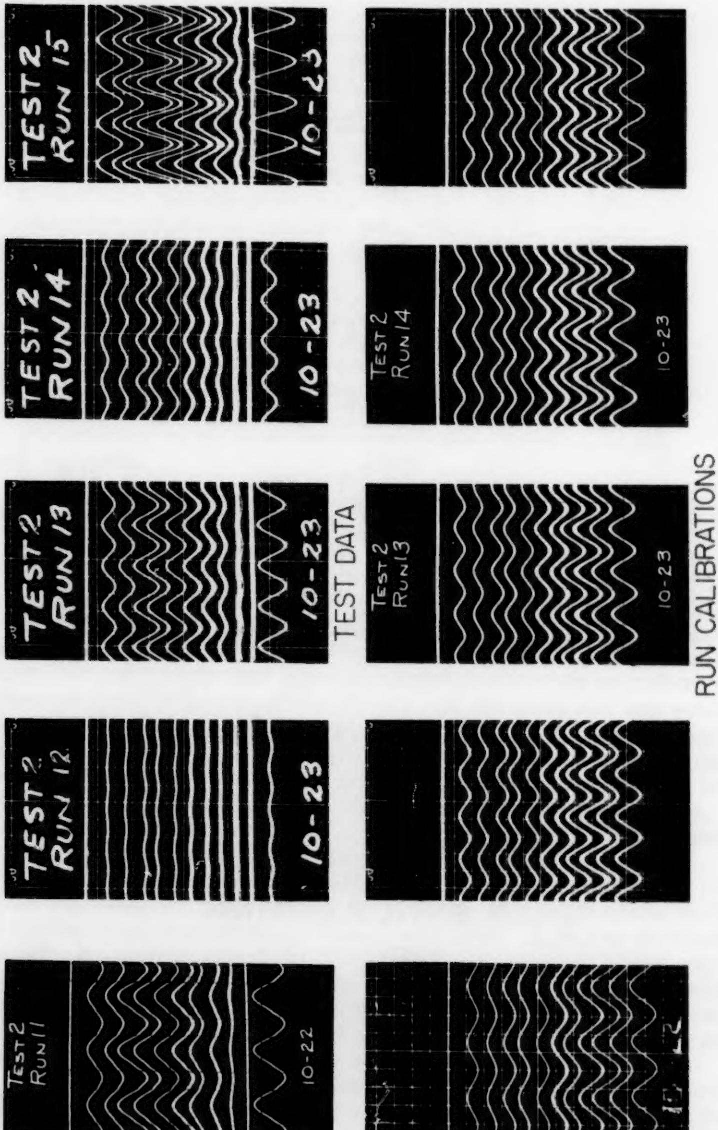
TABLE OF GAGE OUTPUT (VOLTS × 10^2)

	TRIAL					AVG.
	1	2	3	4	5	
P=0	4.436	4.440	4.425	3.951	3.940	4.239
P=50	4.506	4.509	4.495	4.024	4.009	4.309
ΔE	.070	.069	.070	.073	.069	.070
C=ΔE/P	.0014	.00138	.0014	.00146	.00138	.0014

AVG. CALIBRATION FACTOR = 14×10^{-6} VOLTS/LB

STRAIN BAR CALIBRATION

FIGURE 5



DYNAMIC TEST DATA

FIGURE 6

data is a photograph of the run calibrations appropriate to the test data. The same method of trace identification is used for the run calibrations.

Static loading was applied to the pile by adjusting the turnbuckle to shorten the oscillator linkage while the slotted guide was held in a fixed position. Gage voltage outputs were first measured and recorded with the pile in equilibrium position as indicated by the metal pointer. Linkage was then shortened slightly and a second series of gage output measurements made and recorded. This process was carried on until a desired maximum range of loading had been completed.

Four series of dynamic tests and one series of static tests were made. Each series was assigned a test number and a given test included several runs. A run consisted of the data for a specific magnitude of applied load. In tests 3 and 4, an overburden was applied by distributing iron weights on a plywood cover that rested on the mud surface (Fig. 2d). The test series are described as follows:

Test 2: Both static and dynamic testing was performed. All runs were made before an overburden had been placed on the surface.

Test 3: The foundation was loaded with an overburden of 175 pounds or 25 psf during testing.

Test 4: The foundation was loaded with an overburden of 350 pounds or 50 psf (Fig. 2d) during testing.

Test 5: These tests were made under the same conditions as for test 2 except that they were performed after the overburden had been removed.

For the test 2 series, the mud sloped gently from pile to tank edge (Fig. 2c) and for the remaining tests a uniform depression of approximately one inch was noted. This was obviously due to compaction of the medium by the overburden.

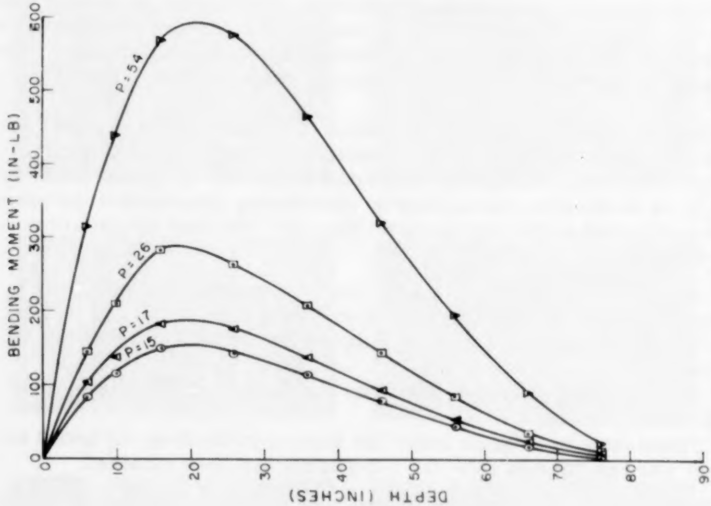
Data Analysis

Analysis of the 35 mm film records was carried out on a "Recordak" film reader. This instrument projected the traces (Fig. 6) on a large ground glass screen which made it possible to measure the curves with a reasonable degree of accuracy. An overlay with a wave height for each gage location was constructed for each set of runs using the appropriate set of run calibration curves. These heights were then compared to the corresponding test run heights and the ratio Δ_b/Δ_c was read directly with ten point dividers. The calibrated value of Δ_c for each location was known so multiplication of this number by the ratio resulted in the maximum moment at locations 1 through 10 and the maximum force at the strain bar. Any phase change between run calibration waves and test run waves was also noted.

The final step in the analysis of the static test data was to determine the change in gage output voltage for each position of the pile, i.e., for each successive applied load, with respect to the outputs recorded at equilibrium. Calibration factors were known for both the pile and strain bar gages so it was then possible to determine the moments or load for each position (Figs. 7, 8 and 9).

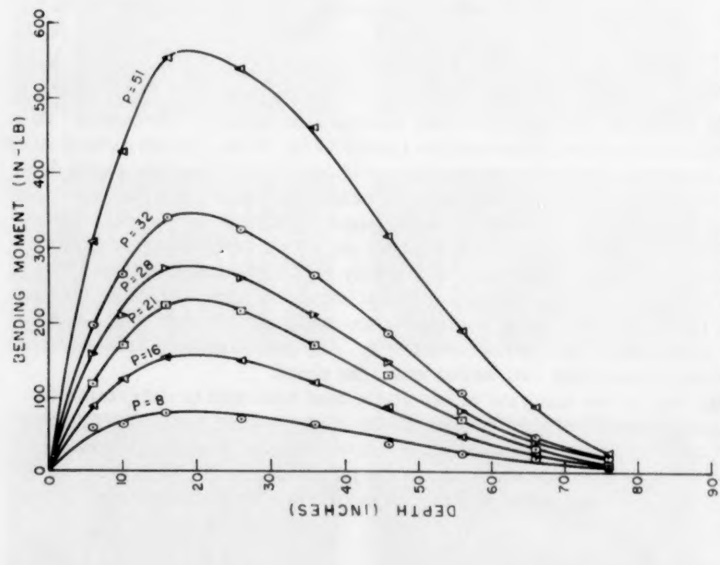
Analytical Solutions

Moment distributions along the model pile were computed by two different



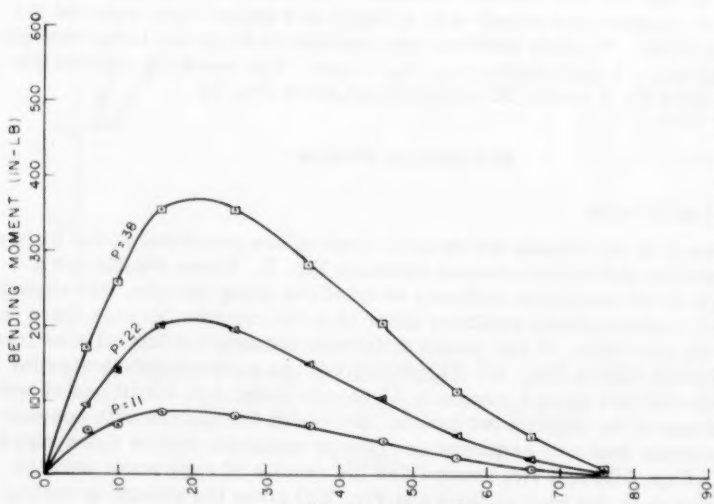
MOMENT DISTRIBUTION UNDER DYNAMIC LOADING (TEST 3)

FIGURE 7(b)



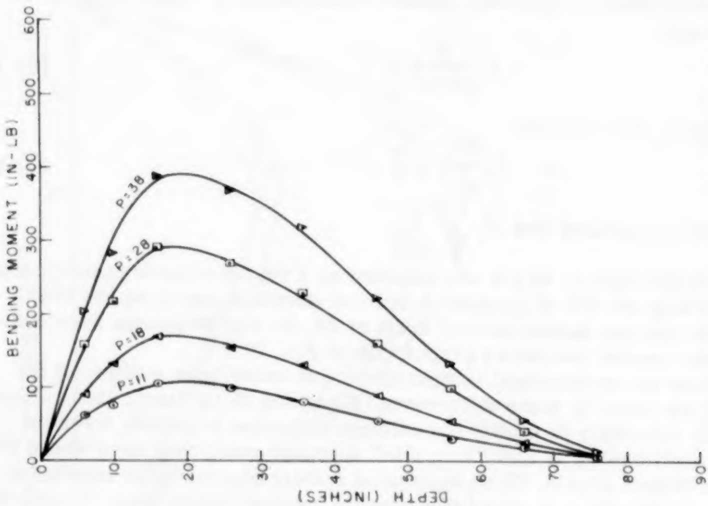
MOMENT DISTRIBUTION UNDER DYNAMIC LOADING (TEST 2)

FIGURE 7(a)



MOMENT DISTRIBUTION UNDER DYNAMIC LOADING (TEST 4)

FIGURE 7(c)



MOMENT DISTRIBUTION UNDER DYNAMIC LOADING (TEST 5)

FIGURE 7(d)

analytical methods, each based upon a different concept of soil modulus. The first method employed was based upon the method presented by Timoshenko (1941) using a constant soil modulus. Modification of the solution to satisfy freehead pile boundary conditions assuming a "long pile" resulted in the following pile deflection formula, referred to an origin of x lodated at the load point of the pile:

$$y = \frac{P}{2EI\beta^3} e^{-\beta x} \cos\beta x \qquad \text{(VI-1)}$$

The moment is then given by

$$M = EI\frac{d^2y}{dx^2} = \frac{P}{\beta} e^{-\beta x} \sin\beta x \qquad \text{(VI-2)}$$

where it will be recalled that $\beta = \sqrt[4]{\frac{E_s}{4EI}}$.

An arbitrary value of 48 psi was selected as a representative constant soil modulus. Using Eq. (VI-2), predicted moment distributions along the length of the model pile for applied lateral loads of 20, 40, and 60 pounds were computed and the results are shown graphically in Fig. 10.

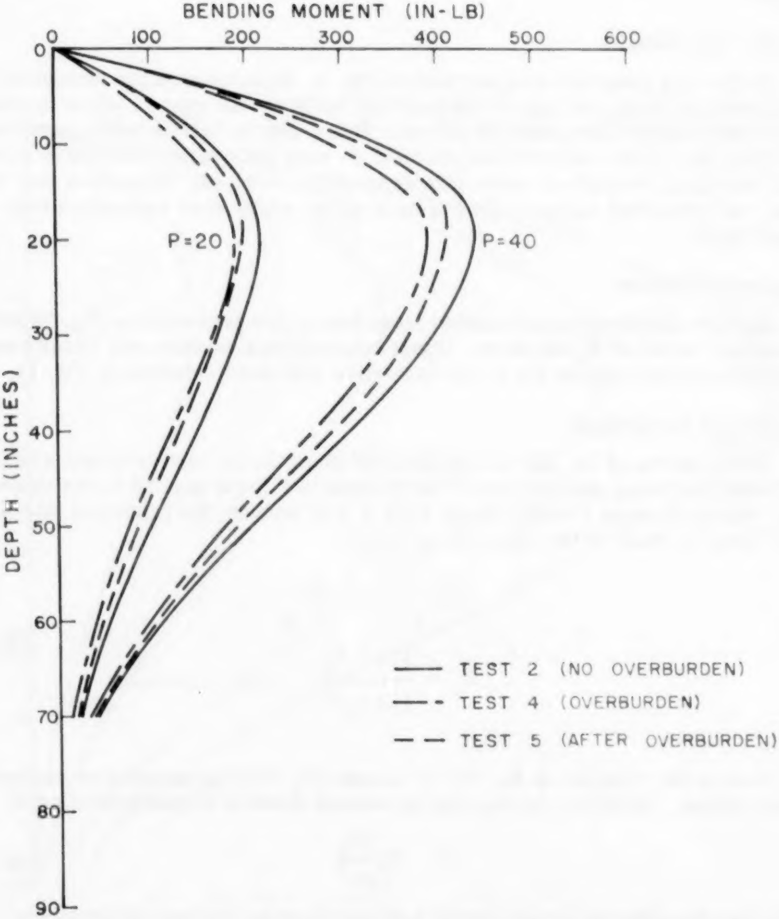
The second set of analytical computations was based upon solution of the pile deflection curve by finite differences (Focht and McClelland, 1955). Here again it was necessary to modify the solution somewhat to satisfy freehead boundary conditions. The four "long pile" boundary conditions used where (1) no shear at bottom of pile, (2) no moment at bottom of pile, (3) no moment at top of pile and (4) shear at top of pile equal to applied lateral load. Boundary condition (3) was the only one that differed from those used by Focht and McClelland. Sixteen increments with a length of 6 inches were selected for the computations. The soil modulus was assumed to be proportional to depth and a value of k = 1 was chosen, i.e., $E_s = x$ psi. The resulting moment distribution curve for a load of 20 pounds is shown in Fig. 10.

Summary of Results

Dynamic Test Results

A summary of the results for dynamic loading are presented in the form of bending moment distribution curves shown in Fig. 7. These results are a comparison of the maximum moments at locations along the pile. No attempt was made to analyze phase positions other than the maxima because there appeared to be very little, if any, phase difference between the lateral load trace and the moment traces (Fig. 6). Fig. 7(a) gives the moment distribution for a range of lateral load from 8 pounds to 51 pounds under soil conditions of test 2. As in some of the subsequent figures, averaging two curves with approximately the same load was employed in order to make the curves more representative. Figs. 7(b) and 7(c) summarize the results of runs made while an overburden was on the mud surface and Fig. 7(d) gives the results of testing after the overburden had been removed.

Intercomparison of dynamic test results is summarized in Fig. 8 for loads of 20 pounds and 40 pounds. These curves were obtained by linear interpolation between appropriate curves on Figs. 7(a) through 7(d). Curves for tests with a 25 psf overburden are not included. It is noteworthy that a definite



REPRESENTATIVE MOMENT DISTRIBUTIONS UNDER DYNAMIC LOADING

FIGURE 8

shift towards less moment occurred for tests with an overburden, and after its removal the moment curves did not shift all the way back to test 2 positions.

Static Test Results

Static load tests are summarized in Fig. 9. Separation of the foundation at the mud line from one side of the pile was noted for all runs in which applied load was greater than about 60 pounds. There was no excess water on the soil surface and it was covered with oil cloth so very little alteration due to wetting or drying took place where pile separation occurred. After each run, the soil was remolded and smoothed to its original shape when separation had been noted.

Analytical Results

Moment distributions calculated from theory are presented in Fig. 10 with assumed values of E_s as shown. Final relationships between test results and analytical computations for a representative load are presented in Fig. 11.

Prototype Predictions

The π terms of Eq. (III-1) may be used to obtain the characteristics of the selected prototype pile for any of the desired test loads applied to the model. For example, using results of test 2 for P = 20 pounds, the prototype lateral load may be found in the following manner.

$$\left(\frac{EI}{P}\right)_p = n^2\left(\frac{EI}{P}\right)_m$$

$$P_p = \frac{(EI)_p P_m}{(EI)_m n^2} = 7580 \text{ pounds} \qquad \text{(VII-1)}$$

None of the π terms of Eq. (III-1) include pile bending moment or maximum fiber stress. However, the moment is related directly to pile deflection:

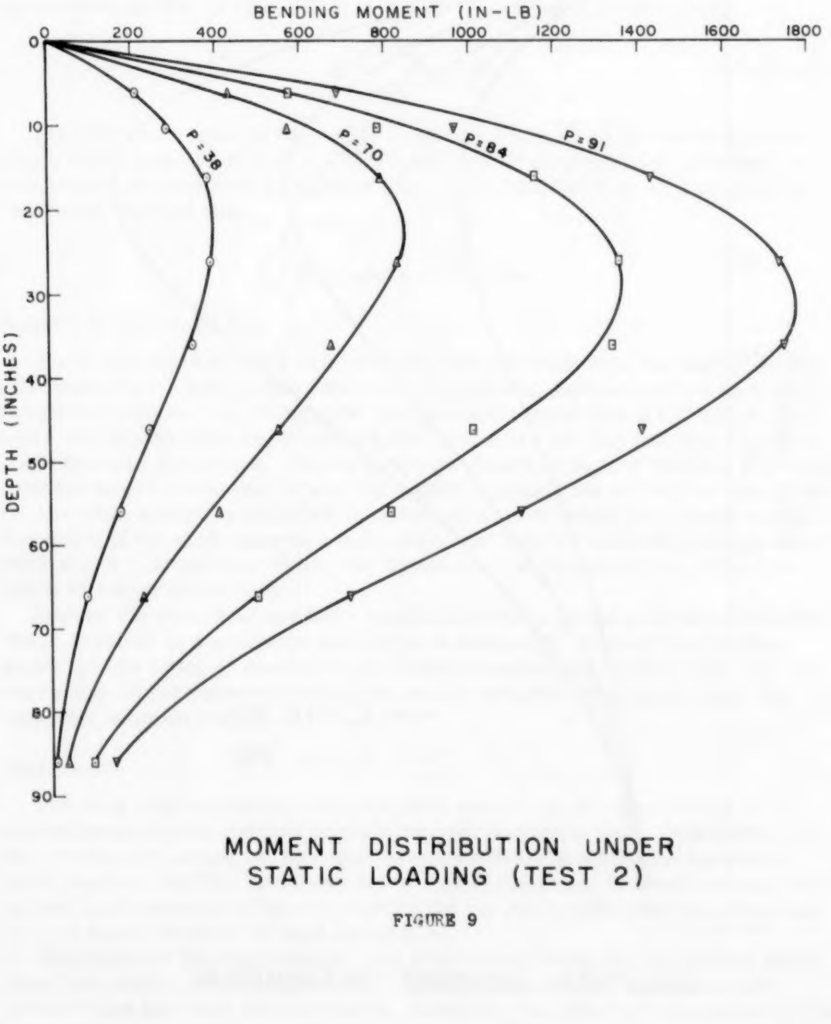
$$M = EI\frac{d^2y}{dx^2} \qquad \text{(VII-2)}$$

This equation applies to both model and prototype so the ratio of prototype moment to model moment may be written:

$$\frac{M_p}{M_m} = \frac{(EI)_p\left(\frac{d^2y_p}{dx_p^2}\right)}{(EI)_m\left(\frac{d^2y_m}{dx_m^2}\right)} \qquad \text{(VII-3)}$$

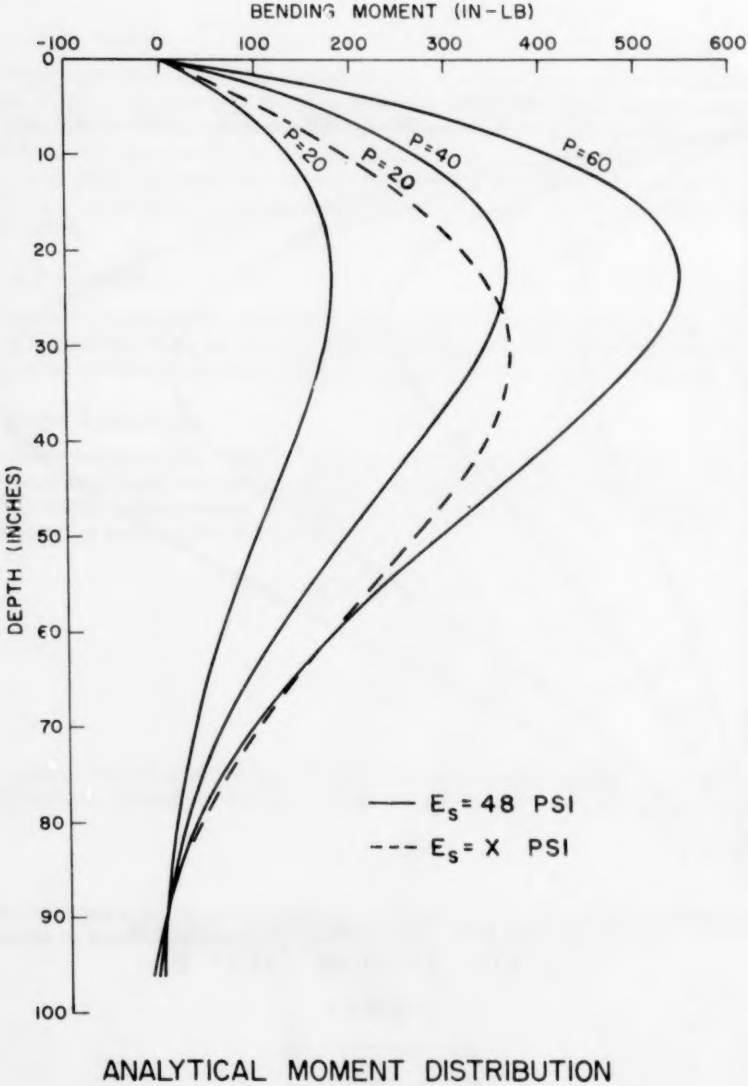
Since x and y are dimensionally equivalent, Eq. (VII-3) may be expressed in the form

$$\frac{M_p}{M_m} = \frac{(EI)_p x_m^2 y_p}{(EI)_m x_p^2 y_m} \qquad \text{(VII-4)}$$



MOMENT DISTRIBUTION UNDER STATIC LOADING (TEST 2)

FIGURE 9



ANALYTICAL MOMENT DISTRIBUTION

FIGURE 10

The model was designed to satisfy the conditions

$$y_p = ny_m \tag{VII-5}$$

$$x_p = nx_m \tag{VII-6}$$

Substitution of (VII-5) and (VII-6) into (VII-4) yields the relationship,

$$M_p = \frac{M_m(EI)_p}{n(EI)_m} \tag{VII-7}$$

The maximum value of M_m = 215 in-lb at a depth of 20 inches as shown in Fig. 8 may be inserted in Eqs. (VII-7) and (VII-6) respectively. The equivalent prototype maximum moment is found to be 102,750 ft-lb at a depth of 25 feet below the mud line.

Discussion of Results

Support at Bottom of Pile

The first series of tests indicated that the pin support at the bottom of the pile might have a significant effect on test results. Because of the very soft foundation medium, the "long pile" design requirement was not fulfilled, i.e., there would have been measurable deflection at the pile tip had there been no pin support to restrain it. The fact that no change of sign of moment occurred over the length of the pile at a given instant supports the contention that without the stud, a slightly different moment adjustment might have taken place. Presence of the stud assured a nodal point for both the moment and deflection curves at x = 96 inches. From the standpoint of a prototype pile, this condition is rather unrealistic.

Both of the analytical methods assumed no shear at the bottom of the pile which resulted in a computed deflection at that point. This deflection was small (on the order of one-thirtieth of the computed deflection at the top) so correction of the boundary conditions would probably not greatly alter the computed moment curves.

Mud Level

The mud level varied for different test series. In the case of test 2, it sloped upward from the tank edges to the pin connected yoke. Application of the overburden forced surface mud to redistribute laterally and assume a level position, but this level was below that of the yoke. Surface rebound occurred upon removal of the overburden but the level remained approximately an inch below the point of load application.

It is doubtful that the lowered mud level alone would have a marked effect upon test results primarily because under normal marine conditions the surface mud has very little strength. However, the effect of compaction in the surface mud is readily ascertained from Fig. 8 where, with loads of 20 and 40 pounds, the test 5 moment curve falls between the test 2 and test 4 curves. If lowered mud level were the only influence, a greater moment would be expected for test 5 than for test 2. Compaction or densification of the mud enabled the soil to take a greater portion of the load, hence, pile bending moment

was less than in the case of test 2.

Calibration

Little variation in voltage output readings occurred in the calibration of strain gages on the model pile. Furthermore, the mean variation of calibration factors was very small so measurements made with these gages should have a high degree of accuracy.

A much greater variation in readings was recorded during the strain bar calibration. It was also found that the readings taken with the bar loaded had a time variation. A second set of strain gages was installed on the bar and variable readings persisted. The reasons for these variations remain unexplained but it appears that the deviations would have very little effect upon dynamic load measurements because the time during load application in one direction would be too short. Under static loading this strain bar "hysteresis" might be significant but checks made during testing revealed only slight drift of the readings and most of this might be explained by relaxation of the foundation.

Characteristics of Measuring Equipment

Irregularities of individual traces or general shifts in position of all traces can be detected on a large portion of the dynamic data (Fig. 6). Oscilloscope distortion, fluctuating line voltages, and effects of varying external fields are largely responsible. To minimize these effects, several pictures were taken for each run and only the best ones used for analysis. In addition, analysis was confined to the wave nearest the middle of each trace. It was found that in spite of general shifts, the relative wave heights between successive crests and troughs did not vary significantly.

The traces are relatively wide. All measurements were made with reference to the top of the curve rather than risking inaccuracy involved in estimating the center of the line.

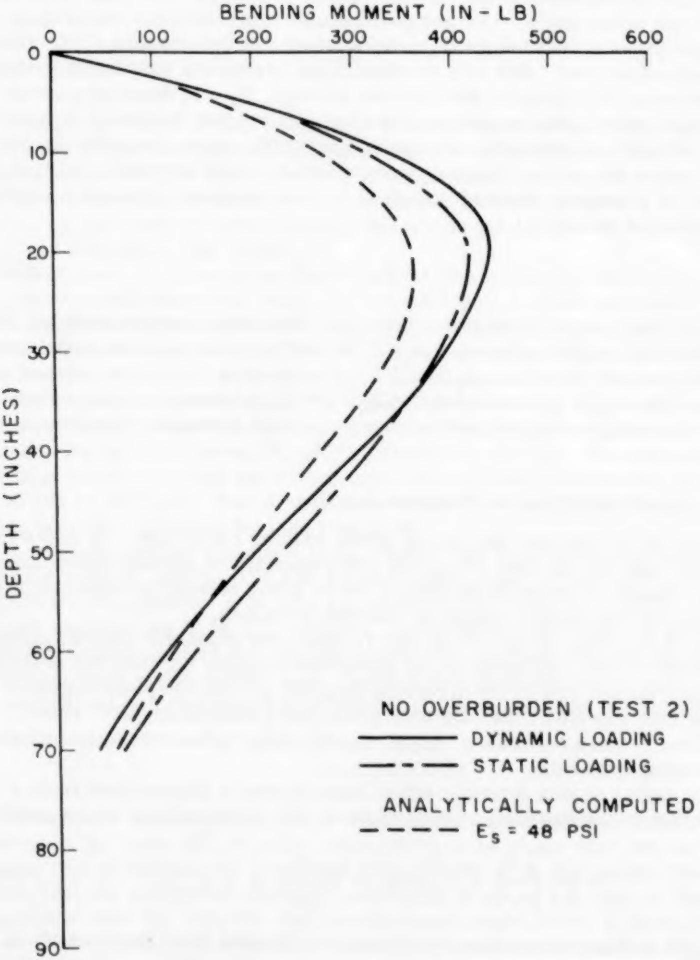
Moment Distribution

For both static and dynamic testing, it was essential to locate the pile equilibrium or "zero stress" position. The pointer greatly aided in establishing the pile equilibrium position with a reasonable degree of consistency. Mislocation of the zero position may account for inconsistency in static load results of test 3.

Increased static load has been previously noted to shift the point of maximum moment downward (Reeves, 1955). Static test results shown in Fig. 8 indicate only a slight shift in moment for moderately large loads. This slight shift may be explained by the fact that such large loads caused a mud separation on one side of the pile and upward heaving on the other. Thus the surface soil capacity had reached a maximum resistance and was transferring the load to greater depths. Had the tank been larger (so that retaining wall restraint would have been less) this shift of the maximum moment downward might have been more pronounced.

No such shift is apparent in the case of dynamic loading. This may be primarily due to the fact that the maximum applied lateral load was about 50 pounds.

The results presented in Fig. 11 are highly significant. It is shown here



COMPARISON OF EXPERIMENTAL TO ANALYTICAL MOMENT DISTRIBUTIONS FOR P = 40 POUNDS

FIGURE 11

that for a representative rate of lateral load oscillation, virtually no difference in maximum bending moment between dynamic and static test results occurred in these tests. The two curves have the same shape and the maximum moment occurs at the same depth. The computed moment distribution based upon E_s = 48 psi gives a reasonable fit to experimentally determined static and dynamic moment curves. For this particular apparatus the equivalent modulus (by areas under the modulus distribution curves), $E_s = x$, does not provide reasonable results. An explanation for this may be that the model soil was not consolidated or compacted enough to justify the assumption that E_s was proportional to the depth. Furthermore, Reeves (1955) has indicated that the "average" of a linearly varying soil modulus should be one-third the modulus at the bottom of the pile, i.e.,

$$\text{Avg. } E_s = \frac{kx_L}{3} \qquad \text{(VIII-1)}$$

where x_L is the length of the pile. Using the assumed constant modulus of 48 psi, it is found that the value of k is 1.5 instead of 1.0 so a more satisfactory relationship might have been $E_s = 1.5\,x$. The greater maximum moment and more rapid decrease to no moment with depth of the experimental curves compared to the analytical curve indicates some actual increase of soil modulus with depth.

Effective Soil Modulus Under Dynamic Loading

It should be pointed out that under static loading conditions, the soft semi-plastic soil has time to flow around or otherwise adjust to pile displacement. This pile relaxation must be accounted for in static testing to eliminate the effects of time dependent changes of induced moments.

Except in the case of extremely low frequencies of lateral load oscillation, "plastic flow" of the soil need not be considered simply because the soil is not continuously loaded in a given direction long enough for the phenomenon to occur. However, motion of the pile within the soil produces another effect which, under certain conditions, might significantly influence the magnitude of pile bending moments.

The character of this dynamic effect may be easily ascertained from a consideration of the equation of motion for a unit length of pile at any depth:

$$(M+M')\frac{d^2y}{dt^2} + \rho\frac{K_D}{2}D\frac{dy}{dt} + EI\frac{d^4y}{dx^4} + E_s y = 0 \qquad \text{(VIII-2)}$$

where K_D is a dimensional drag coefficient of viscous fluid flow and M' is added mass per unit length due to soil inertia (see also Wilson, 1941). If it is assumed that the lateral velocity of the pile through the soil is extremely small, the second term of Eq. (VIII-2) may be neglected. Letting $y = X \sin\omega t$, where X is some function of depth, the equation becomes

$$-(M+M')X\omega^2\sin\omega t + EIX^{IV}\sin\omega t + E_s X\sin\omega t = 0 \qquad \text{(VIII-3)}$$

which simplifies to

$$EIX^{IV} + (E_s - M_e\omega^2)X = 0 \qquad \text{(VIII-4)}$$

where M_e is the effective mass of the pile, i.e., the sum of the actual mass, M, and the mass of the displaced soil, M'. It may be readily observed that since X is equal to $y/\sin\omega t$, the above equation is of the same form as Eq. (II-2):

$$EI\frac{d^4y}{dx^4} + E_s y = 0 \tag{II-2}$$

Hence, Eq. (VIII-4) may be regarded as the basic differential equation governing pile deflection induced by a dynamic lateral load. This equation differs from (II-2) in that the dynamic soil modulus is the difference between the static modulus and a term which depends upon the frequency of load oscillation. In the case of static loading, the frequency is zero and the dynamic equation reduces to Eq. (II-2).

Based upon the concept developed above, the model for this study may be employed to illustrate the change in effective soil modulus produced by a relatively low frequency load oscillation as compared to static load conditions. Assuming a one foot length of pile and a soil specific weight of 70 pcf, the mass of displaced soil is found to be 0.067 slugs per foot. The mass of the pile is 0.039 slugs per foot and thus the effective mass, M_e, becomes 0.106 slugs per foot. Given that frequency of load oscillation is 6.28 radians per second, the dynamic term, $M_e\omega^2$, becomes only 4.2 psf. The assumed static modulus which provided analytical moments in close accord with test results is 48 psi or 6910 psf. The dynamic modulus is consequently found to differ from the static modulus by less than 0.1%.

A word of caution may here be in order since the mud is certainly a viscous medium and Eq. (VIII-2) neglects any effect of viscous damping or soil shear resistance. It is also possible that because the medium is more nearly elastic than fluid, the effective mass of the pile might be much larger than the simple sum of pile and displaced soil masses. The dynamic soil modulus could consequently vary considerably from the static modulus which is in contrast to conclusions that might be drawn from the above computations.

CONCLUSIONS

This study indicates that a pile subjected to an oscillating lateral load vibrates in the form of a standing wave which is in phase with the oscillating load. This condition was predicted from theory when the assumption was made that the velocity of the pile, and hence damping capacity of the soil, was negligible (see Eq. VIII-2). Had damping been significant, a phase displacement would have occurred between the load oscillation and induced moments along the pile.

Test results indicate that for a relatively low frequency of load oscillation, no dynamic load factor is required to produce the same maximum subsurface bending moment in the pile under static and dynamic loading. However, there is the possibility that the system may have a natural frequency of vibration in which case resonance with the forcing frequency could produce dynamical bending moments in excess of static. The determination of a reliable dynamic load factor (or factors) would require testing over a wide range of forcing frequencies and types of foundations.

With experimental data provided by this study, a reasonable fit may be made of dynamic test results to moment distributions computed from theory assuming a constant soil modulus.

These tests suggest that the depth at which maximum bending moment occurs is not dependent upon magnitude of lateral load unless pile deflection becomes large enough to stress the soil beyond its elastic range.

Overburden placed on the soil surface has the effect of reducing pile bending moment. The shape of the moment curve remains about the same and maximum moment occurs at approximately the same depth as in the case of no overburden.

ACKNOWLEDGEMENTS

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DEWATERING EXCAVATION, LOW SILL STRUCTURE, OLD RIVER, LA.

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(Proc. Paper 1536)

SYNOPSIS

A system of large-diameter deep wells was installed in the excavation for the low sill structure for Old River control for the purpose of lowering the hydrostatic head in a deep stratum of pervious sand which lies beneath the excavation for the structure. The specifications for the project required that the number, arrangement, and capacity of the wells be such as to make possible the reduction in hydrostatic head in the deep sands to a level at least 5 ft below the bottom of the excavation (el—15 msl) with a river stage of 60 msl. Final approval of the deep well system was contingent upon proven performance by field tests and evaluation. To determine the adequacy of the deep well system, a series of comprehensive tests was made; the test procedures used and evaluation of the data are described in this paper.

From the pumping tests it appears that the 20 temporary wells and 7 permanent wells are adequate for controlling hydrostatic pressures in the deep sands beneath the main portion of the excavation for river stages up to el 60. Adequate pressure reduction cannot be achieved in areas of riprap extensions with the existing well system and additional temporary wells will have to be installed in these areas. A well flow of about 16,000 gpm is indicated for a river stage of 60 msl and the excavation to final grade. This corresponds to an average flow of about 600 gpm per well. The well flow per foot of net head on a well system is about 175 gpm and about 185 gpm per foot of average drawdown. The permeability of the deep sand aquifer as computed from the pumping tests was about 1100 x 10^{-4} cm per sec. Lowering of the hydrostatic head in the deep sand stratum below the bottom of the excavation also has had a material effect on drying the excavation slopes and the bottom

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of the excavation.

INTRODUCTION

The low sill structure, a controlled spillway, is one of the principal features of the Old River control project planned for control of flow of the Mississippi River into the Atchafalaya River. The structure is located on the west bank of the Mississippi River about 35 miles south of Natchez, Mississippi. It will be constructed of reinforced concrete and will contain vertical lift steel gates. It will have a gross length of 566 ft between faces of the abutment training walls and will have 11 gate bays. Four bays on each end of the structure will have a weir crest elevation of +10 ft msl; the three central bays of the structure will have a weir crest of -5 ft msl. The gated portion of the structure and the abutment piers will be founded on piles. Seven permanent relief wells have been installed in the stilling basin area to provide relief of excess substratum pressures that will develop in a pervious sand stratum beneath the structure when it is placed in operation.

The excavation required for the low sill structure is approximately 1500 ft long and 600 to 1000 ft wide, with a depth ranging from 55 to 70 ft below the ground surface; el 48. Extensions in each corner about 450 ft long will be excavated after a major portion of the structure proper is completed. Riprap will be placed on the slopes of these extensions to form a part of the slope protection for the approach and outlet channels. The excavation involves the removal of clays, silts, and silty sands. The bottom of the excavation is underlain by alternating strata of silts, sandy silts, and silty sands with some interspersed clay strata to a depth of 50 to 60 ft. These strata in turn are underlain by a pervious stratum of sands which has a thickness of 40 to 60 ft and a coefficient of permeability of about 1100×10^{-4} cm per sec. The sands are underlain in turn by stiff Tertiary clays. A section through the excavation taken perpendicular to the Mississippi River and depicting soil conditions at the site is shown on Fig. 1.

Readings from piezometers installed in the deep sand stratum in 1954 show that the hydrostatic pressure in this stratum reflects closely the stage of the Mississippi River. A maximum river stage of el 60 during construction would create a net head of approximately 70 to 80 ft beneath the excavation; such a differential head would probably cause sand boils and could cause a blowup in the bottom of the excavation unless relieved. Not only was it necessary to relieve excessive artesian pressures in the deep sands but also to lower the water table in the deep sand below the bottom of the excavation in order to achieve dry working conditions. Therefore, a system of large diameter wells was installed to lower the hydrostatic head in the sands underlying the excavation. A system of shallow wellpoints was also installed on the slopes of the excavation to intercept seepage which might otherwise emerge. After the excavation is to grade, the maximum head in the deep sands is to be maintained at least 5 ft below the bottom of the excavation for any river stage up to el 60. After the drains, base slabs, impervious blankets, and riprap have been placed, the hydrostatic head in the deep sands may be allowed to rise to an elevation not more than 5 ft above the surface of the riprap or concrete or tailwater as the structure or excavation is flooded.

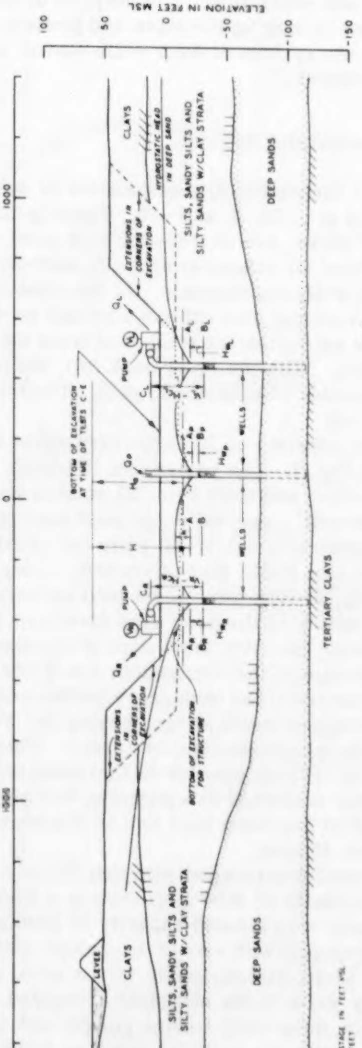

Fig. 1. Soil conditions at site and notations for analysis of pumping test data

Final approval of the deep well system was contingent upon proven performance by actual field tests and evaluation. The purpose of this paper is to summarize the procedures used in making the tests and present the specific purpose of each test, describe the system of deep wells tested, and present the results and conclusions obtained.

Dewatering System

The wellpoint system on the excavation slopes consists of wellpoints connected to header pipes installed at el 20, 5, and -10. These points have a depth of 25 ft below the header pipes, are surrounded with sand, and are spaced on 8-, 7-, and 6-ft centers for stages at el 20, 5, and -20, respectively. No tests were made on the wellpoint systems. At the time of tests on the deep wells, the first-stage system was shut off and although no actual measurements were made it is estimated the total flow from the second-stage system does not exceed 200 gpm. With the river bank full, the total flow from the wellpoint system is not expected to exceed 500 gpm. The third-stage system has not yet been installed.

The deep dewatering system consists of 20 temporary wells and 7 permanent wells located as shown in Fig. 2. The permanent (W) wells are located across the middle of the excavation and have 8-in. ID wooden risers and slotted wooden screens surrounded with a specially designed sand-gravel filter 6-in. thick. The slots in the screens are 3/16 in. wide and provide an open area of approximately 30 sq in. per linear foot of screen. The gradation of the filter gravel is shown in Fig. 3. The permanent well screens are 32 ft long and are placed about the middle of the deep sand stratum. Nine of the temporary wells are located along the riverward edge of the excavation, 7 are located along the landward edge of the excavation, and 4 are located in the areas of abutments for the structure. The temporary wells consist of 8-in. ID steel riser pipe and wire-wrapped metal screen having No. 55 slots which provide an open area of 87 sq in. per linear foot of screen. These screens are surrounded with a sand-gravel filter similar to that used for the permanent relief wells. Each well was subjected to a pumping test shortly after installation. The specific yield of the wells (per foot of drawdown) ranged from 31 to 67 gpm and averaged 45 gpm.

The temporary wells riverward are pumped with two 12-in. centrifugal wellpoint pumps with a rated capacity of 4500 gpm each at a 20-ft vacuum and a 10-in. centrifugal standby pump with a rated capacity of 2700 gpm. The temporary wells landside are pumped with two 12-in. pumps with a rated capacity of 4000 gpm each and a 10-in. standby pump with a rated capacity of 2700 gpm. The four temporary wells in the abutment areas and the 7 permanent wells are pumped with 8-in. deep-well turbine pumps with a rated capacity of 600 gpm each; these pumps can pump as much as 800 or 900 gpm by increasing the speed above normal.

Description of Pumping Tests

A number of pumping tests were performed to check the performance and adequacy of the deep dewatering system. These tests included the measurement of flow from various groups of wells and the hydrostatic head at certain points under and in the area of the excavation. The locations of piezometers

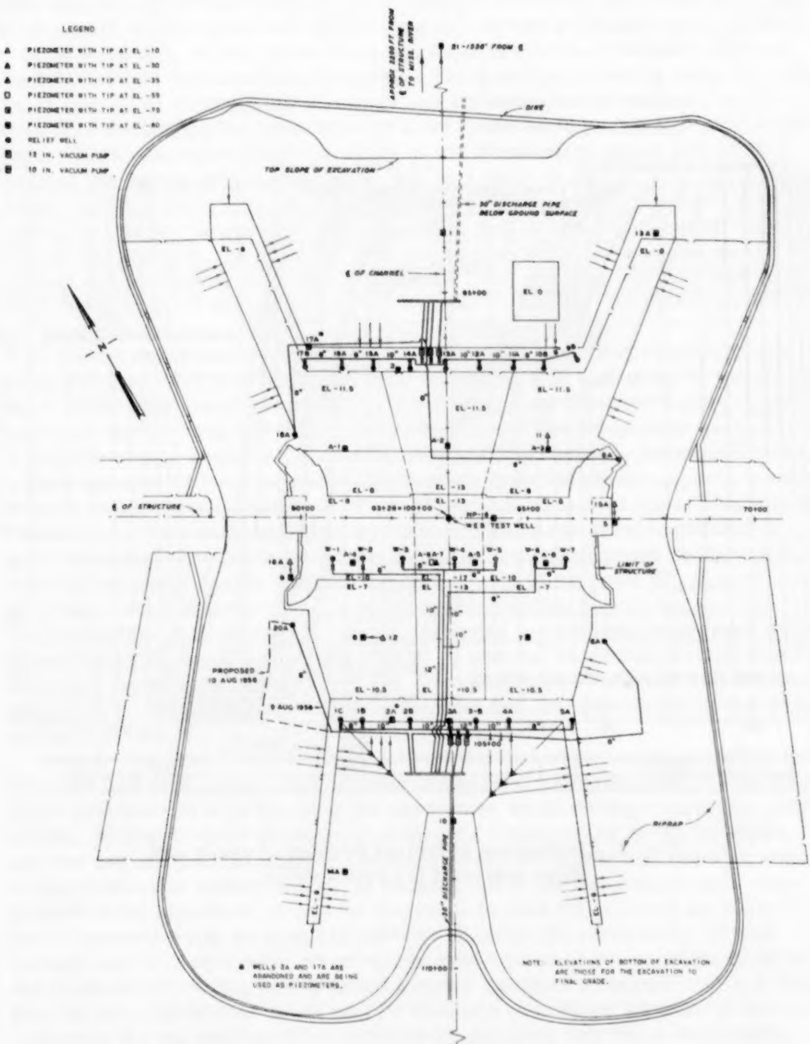

Fig. 2. Plan of excavation, dewatering system, and piezometers

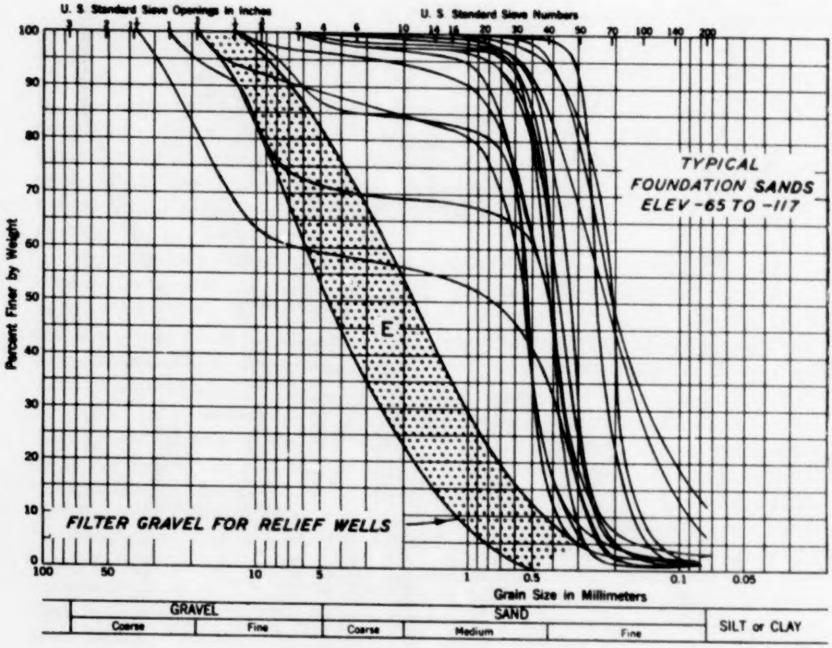

Fig. 3. Gradations of typical foundation sands and filter material for relief wells

used for measuring the hydrostatic head are shown in Fig. 2. An outline of the tests performed is given in table 1. Tests A and B were made in May 1956 with the excavation at el 20, primarily to determine the adequacy of the wells installed and connected at that time for carrying the excavation down to el 5. A summary of data from these two tests is contained herein, although the detailed test data are not presented. The principal pumping tests (C_1, C_2, C_3, D, and E) to evaluate the adequacy and performance of various combinations of wells with the excavation to final grade were performed on 8 and 9 August 1956. On these dates the stage of the Mississippi River was at 18.3 msl and the bottom of the excavation was at 5 msl. In test C_1, all of the wells shown in Fig. 2 were pumped. In all tests except C_2 and C_3 the vacuum at the centrifugal pumps connected to the riverside and landside temporary wells and the drawdown in the abutment and permanent wells were set so as to achieve approximately the same hydrostatic head immediately adjacent to all wells being pumped during the tests. In tests C_2 and C_3 the water in the permanent wells was pumped below that in the other wells.

In setting the vacuum for the centrifugal pumps and the drawdown in the wells being pumped with turbine pumps, consideration was given to the anticipated flows from the different wells, the elevation of the centrifugal pumps, head loss in the wells and in the collector pipe for the temporary wells.

A preliminary analysis of the data obtained from test C_1 indicated that the system pumped in the above manner might not provide adequate pressure reduction with the excavation to final grade and a Mississippi River stage of 60. Therefore, it was decided to run additional tests to see what improvement could be achieved in performance by pumping the 7 permanent wells at somewhat higher rates of flow than during test C_1. In these tests (C_2 and C_3) the permanent wells were pumped at rates approximately 55 percent and 115 percent more than during test C_1. (It was intended to pump the temporary wells at increased rates of flow of only 20 and 40 percent more than during test C_1. However, in changing the speed of the deep well pumps, another unintentional change was made which significantly changed the characteristics of the pump so that the flow was increased more than intended.)

Test D was performed to determine the amount of pressure reduction that could be achieved by pumping only the temporary wells, in event it should become desirable to stop pumping the permanent wells during concreting operations. In test E, the 7 permanent wells, the 4 temporary abutment wells, and the end and middle wells of the lines on the riverside and landside ends of the excavation were pumped. In backing out of the excavation upon completion of the structure, it will be desirable to shut off and plug as many of the temporary wells as possible prior to flooding the excavation. It was thought that with favorable river stages (and season of the year) pumping of the wells tested during test E would provide adequate pressure relief during backing out operations. As it may be possible to achieve adequate pressure reduction during backing out operations by pumping only the 4 temporary abutment wells and the 7 permanent wells, theoretical computations were made, based on a flow of 600 gpm from each well, to determine the maximum pressure reduction that could be achieved by pumping only from these wells. This condition has been designated F in table 1 and in the remainder of this paper.

The hydrostatic pressures in the deep sands were obtained from the piezometers located as shown in Fig. 2. These piezometers consisted of brass

Table 1

Summary of Pumping Tests on Deep Dewatering System

Low Sill Structure

Test	Wells Pumped	Water Level in Wells	Date	Miss. R. Stage msl	Bottom of Excavation msl	Average Head on Well System ft	Avg Max Head in Deep Sand beneath Excavation msl	Total Well Flow gpm	Total Well Flow per ft Net Head on Well System gpm	Remarks
A	9 RS temporary wells 6 LS temporary wells	Approximately level at El -1.9	16 May'56	30.7	20	31.4	9.2	4630	147	
B	9 RS temporary wells 7 LS temporary wells 7 permanent wells	Approximately level at El -2.5	31 May'56	26.3	20	28.2	3.6	5260	185	
C_1	9 RS temporary wells 7 LS temporary wells 4 temporary (abutment) wells 7 permanent wells	Approximately level at El -16.5	8 Aug'56	18.3	5	34.0	-10.4	6000	176	
C_2	9 RS temporary wells 7 LS temporary wells 4 temporary (abutment) wells 7 permanent wells	Approximately level except lower along permanent wells El -17.1 at temporary wells El -21.5± at permanent wells	8 Aug'56	18.3	5	34.9	-12.8	6310	181	Permanent wells pumped at a rate approximately 55% more than in test C_1.
C_3	9 RS temporary wells 7 LS temporary wells 4 temporary (abutment) wells 7 permanent wells	Approximately level except lower along permanent wells El -18.6 at temporary wells El -26.5± at permanent wells	8 Aug'56	18.3	5	36.2	-14.6	6620	183	Permanent wells pumped at a rate 115% more than in test C_1.
D	9 RS temporary wells 7 LS temporary wells 4 temporary (abutment) wells	Approximately level at El -14.9	9 Aug'56	18.3	5	32.1	-7.2	5440	169	
E	3 RS temporary wells 3 LS temporary wells 4 temporary (abutment) wells 7 permanent wells	Approximately level at El -15.2	9 Aug'56	18.3	5	32.5	-8.0	5012	154	
F	4 temporary (abutment) 7 permanent wells	Approximately level at El -16.8		38.0*	-15±	53.4	-0.4**	6600	124	Based on theoretical computations

* Maximum river stage without exceeding a flow of 600 gpm from permanent wells.
** At piezometer A-2.

well strainers 24 in. long with no. 25 slots attached to 1-1/4-in.-diameter riser pipes. The piezometers were read after the piezometric pressure became stabilized for the various conditions. (Stabilization of the hydrostatic head in the deep sands usually occurred in about 30 minutes.) The piezometric data obtained for the various tests are tabulated in table 2.

Flow from the various groups of wells being pumped was measured by a pitometer inserted in the discharge lines from the various wells. In most instances the velocity in the pipe was determined at several points across the diameter and the average velocity determined graphically from plots of these measurements. In some instances the velocity was measured only at the center of the pipe and the average velocity was obtained by multiplying that at the center of the pipe by the appropriate pipe coefficient shown in the following tabulation.

Type Pump	Pipe Diameter in.	Pipe Coefficient
Centrifugal	10	0.82
	12	0.80
Deep well turbine	8	0.86
	10	0.76
	12	0.76
Average		0.82

The above pipe coefficients are average values obtained from data where the velocity was measured at several points in the pipe. The reason for the variations in values is not known, but may possibly be attributed to air bubbles in the flow from the centrifugal pumps, which were operating at very high vacuums, and/or inaccuracies of readings resulting from surging in the discharge line.

Observed hydrostatic heads at the various piezometers and at wells not being pumped are given in table 2 for tests A-E. Observed well flows and hydrostatic heads beneath the excavation are averaged and summarized in table 3. The flows from the various groups of wells for the different tests are given in table 4. Observed hydrostatic heads at the various piezometers and flow from the deep wells have been plotted in Fig. A-1 through A-5 in Appendix A for pumping tests C-E.

Analyses of Test Data

Nomenclature, Head Losses in Wells and Collector Pipes, and Piezometric Levels

Before it was possible to determine the true characteristics of the well systems tested, it was necessary to determine the head loss in the various wells being pumped and in the collector pipes for the temporary riverside and landside wells. The head loss through the gravel filter and wooden well screens was estimated from laboratory and field measurements[1,2] for similar filters and slotted screens. The head loss through the gravel filter and metal well screen were determined from a pumping test on temporary well 11-A. The head losses within the well screen, riser pipe, and fittings for the wells connected to the centrifugal pumps were computed from pipe flow formulas, as were the head losses in the collector pipes for the pumps.

Table 2

Piezometer and Well Readings for Pumping Tests on Deep Dewatering System

Low Sill Structure

Piezometer Number	Tip Elevation	TEST A	B	C_1	C_2	C_3	D	E
A 1	-80	---	3.3	-10.6	-12.3	-13.6	-7.6	-8.0
A 2	-80	8.0	1.9	-10.7	-11.2	-14.4	-7.3	-7.9
A 3	-80	8.6	3.2	-9.9	-11.6	-13.6	-8.0	-7.2
A 4	-80	---	0.4	-12.5	-16.0	-18.8	-6.6	-11.7
A 5	-80	---	0.3	-12.8	-16.1	-19.1	-7.2	-10.7
A 6	-55	---	0.3	-12.4	-15.7	-18.7	-7.1	-10.4
A 7	-35	---	2.2	-10.4	--	-13.1	-6.0	-7.7
A 8	-80	9.4	0.7	-12.6	--	-20.0	-7.0	-10.1
1	-80	12.0	7.9	-4.0	--	-5.6	-2.6	-1.6
2	-80	6.0	2.2	-10.6	-11.5	-12.3	-8.7	-6.1
5	-80	10.8	4.4	-9.9	-12.4	-14.3	-6.8	-8.0
6	-80	11.4	4.5	-10.2	--	-12.1	-1.6	-7.7
7	-80	9.0	3.1	-11.3	-14.2	-16.3	-7.3	-8.8
8	-80	9.3	2.6	-10.6	-13.0	-14.7	-7.0	-8.1
10	-80	10.4	5.9	--	--	--	--	--
11	-30	10.9	7.4	-4.2	--	-3.9	-2.4	-2.6
12	-30	8.4	2.6	-9.7	--	-12.0	-6.4	-7.4
21	-80	21.1	17.3	+7.0	--	+6.5	+7.8	+8.0
22	-80	20.5	17.0	+6.9	--	+6.6	+7.7	+7.6
2 A***	Well	--	4.9	-10.6	--	-10.3	-9.1	-8.7
13 A	-80	14.4	10.4	-1.2	--	-2.8	+0.1	+0.7
14 A	-80	14.1	9.7	-2.2	--	-3.8	-0.5	+0.1
15 A	-10	3.3	5.0	+7.2	--	+7.1	+6.9	+6.5
16 A	-10	7.4	5.1	--	--	--	-3.1	-3.1
17 A***	Well	8.7	4.8	-8.0	--	-9.8	-6.2	-5.5
MP 2	-18	26.3	22.6	+13.1	--	+13.3	+13.5	+13.5
MP 7	-5	22.1	18.4	+7.1	--	+7.6	+8.4	+7.4
NP 16	-75	8.9	1.0	--	--	--	--	--
WES well		8.9	0.9	-12.2	-15.3	-18.1	-7.2	-9.8
Wells								
W 1		10.4	-2.0	-16.3	-18.3	<-22.8	-6.1	-14.5
W 2		10.8	-2.0	-16.3	<-22.0	<-22.8	--	--
W 3		9.0	-2.0	-16.3	-22.6	<-22.6	-7.2	-14.5
W 4		9.2	-2.0	-16.3	--	<-22.2	-7.2	--
W 5		9.2	-2.0	-16.3	<-22.6	<-22.6	--	--
W 6		9.9	-2.0	-16.3	--	<-22.3	-7.2	-14.4
W 7		--	-2.0	-16.3	-20.2	<-22.2	-7.0	-14.4
6 A		--	--	-16.3	-16.4	<-18.2	-14.3**	-14.3
8 A		--	--	-16.3	--	-18.4	-14.3**	--
18 A		--	--	-16.3	--	-18.6	-14.3**	-16.5
20 A		--	--	-16.3	-17.0	-19.3	-14.3**	--
RS Pump								
Vacuum*		-27.2	-26.9	-26.1	-26.8	-27.5	-24.9	-25.5
El pump		24.0	24.0	+8.0	+8.0	+8.0	+8.0	+8.0
LS Pump								
Vacuum*		-28.0	-27.9	-26.5	-27.3	-28.0	-24.9	-23.8
El pump		23.9	23.9	+9.5	+9.5	+9.5	+9.5	+9.5
Bottom of excavation		20	20	+5	+5	+5	+5	+5
Mississippi River		30.7	26.3	18.3	18.3	18.3	18.3	18.3
Date		16 May	31 May	8 Aug	8 Aug	8 Aug	9 Aug	9 Aug

* Vacuum in feet of water.
** Approximate.
*** Abandoned well.
Below elevation given.
Note: Piezometer and well readings are in feet msl.

Table 3

Observed Well Flows and Hydrostatic Heads Beneath Excavation
for Various Combinations of Wells

Low Sill Structure

Test or Well Combination	Hydrostatic Head or Elevation						
	A	B	C_1	C_2	C_3	D	E
Mississippi River Stage, msl	30.7	26.3	18.3	18.3	18.3	18.3	18.3
Average elevation water in wells:							
RS (temporary) wells	-1.3	-2.1	-17.3	-18.2	-19.1	-15.8	-16.9
LS (temporary) wells	-2.9	-3.5	-16.4	-17.3	-18.1	-14.6	-13.8
Abutment (temporary) wells	*	*	-16.3	-16.7	-18.7	-14.3 ±	-15.4
Permanent wells	9.7*	-2.0	-16.3	-21.5±	-26.5±	-7.2 *	-14.4
Average for excavation**	-1.9	-2.5	-16.5	-17.1	-18.6	-14.9	-15.2
Average hydrostatic head at periphery of well , msl:							
RS (temporary) wells	-0.2	-1.5	-16.6	-17.7	-18.7	-15.1	-15.3
LS (temporary) wells	-1.5	-2.7	-15.6	-16.6	-17.5	-13.6	-12.5
Abutment (temporary) wells	*	*	-15.1	-15.7	-17.7	-12.8 ±	-14.2
Permanent wells	9.7	-1.6	-15.8	-19.9	-25.9	-7.2 *	-14.3
Average for excavation**	-0.7	-1.9	-15.7	-16.6	-17.9	-13.8	-14.2
Average net head on wells, ft	31.4	28.2	34.0	34.9	36.2	32.1	32.5
Piezometer Group I	A2, A3, A8 7, 8, WES W1, W4, W6	5, 6	A1, A2, A3 5, 6, 7, 8	A1, A3 7, 8	A1, A3 7, 8	A1, A2, A3 WES, 7, 8 W1, W4, W7	A1, A2, A3 5, 6, 7, 8
Piezometer Group II	-	A1, A3	-	-	-	-	2
Piezometer Group III	-	7, 8	-	-	-	-	-
Average piezometer readings, msl:							
Piezometer Group I	9.2	4.5	-10.4	-12.8	-14.6	-7.2	-8.0
Piezometer roup II	-	3.3	-	-	-	-	-6.1
Piezometer Group III	-	2.9	-	-	-	-	-
Piezometer 13A	14.4	10.4	-1.2	-	-2.8	+0.1	+0.7
Average for excavation	9.2	3.6	-10.4	-12.8	-14.6	-7.2	-8.0
Average maximum (net) piezometric head, ft:							
Piezometer Group I	9.9	6.4	5.3	3.8	3.3	6.6	6.2
Piezometer Group II	-	5.2	-	-	-	-	8.1
Piezometer Group III	-	4.8	-	-	-	-	-
Piezometer 13A	15.1	12.3	14.5	-	15.1	13.9	14.9
Average for excavation	9.9	5.5	5.3	3.8	3.3	6.8	6.2
Average maximum (net) piezometric head, per cent:							
Piezometer Group I	31.5	22.6	15.7	10.9	9.1	20.5	19.1
Piezometer Group II	-	18.4	-	-	-	-	24.9
Piezometer Group III	-	17.0	-	-	-	-	-
Piezometer 13A	48.1	43.6	42.7	-	41.8	43.1	45.9
Average for excavation	31.5	19.5	15.7	10.9	9.1	20.5	19.1
Average total drawdown, ft	26.4	25.4	31.4	32.9	35.3	29.3	29.4
	Well Flow in gpm						
Well flow per ft of average drawdown	176	205	191	192	188	186	170
Well flow per ft of net head on wells	147	185	176	181	183	169	154

* Wells not pumped.

** Average of temporary wells 1-C, 3-A, 5-A (LS); 9-B, 13-A, 17-B (RS); abutment wells 6-A, 8-A, 18-A, 20-A; permanent wells W-1, W-4, W-7; (if pumped). Elevation of water in and hydrostatic head at periphery of permanent wells not included for tests C_2 and C_3.

Table 4

Well Flows During Pumping Tests on Deep Dewatering System

Low Sill Structure

Wells	A	B	C_1	C_2	C_3	D	E	F*
Riverside Wells								
Total flow	2580	1690	1705	1493	1280	1915	1175	0
Number of wells	9	9	9	9	9	9	3	0
Average flow per well	287	188	190	166	142	213	392	0
Landside Wells								
Total flow	2050	1560	1645	1503	1360	1920	1050	0
Number of wells	6	7	7	7	7	7	3	0
Average flow per well	342	223	235	215	195	274	350	0
Wells 6A and 8A								
Flow - Well 6A	0	0	176	174**	172	270	216	600
Flow - Well 8A	0	0	314	299**	284	382	348	600
Total	0	0	490	473**	456	652	564	1200
Average flow per well	0	0	245	236**	228	326	282	600
Wells 18A and 20A								
Flow - Well 18A	0	0	426	365**	304	430	435	600
Flow - Well 20A	0	0	446	438**	430	525	440	600
Total	0	0	872	803**	734	955	875	1200
Average flow per well	-	-	436	402	367	478	438	600
Total - Abutment Wells	0	0	1362	1276	1190	1607	1439	2400
Permanent Wells								
Flow - Wells W1 + W2	0	554	426	685**	945	0	447	1200
Flow - Wells W3 + W4 + W5	0	1060	490	728**	965	0	519	1800
Flow - Wells W6 + W7	0	358	378	630**	882	0	382	1200
Total	0	1972	1294	2043**	2792	0	1348	4200
Average flow per well	-	282	185	292**	399	0	193	600
Total flow from wells	4630	5220	6000	6310**	6620	5440	5012	6600
Average head on wells	31.4	28.2	34.0	34.9	36.2	32.1	32.5	53.4***
Well flow per ft head	147	185	176	181	183	169	154	124 ***

* Assumed values.

** Not measured. Average of measured flows for tests C_1 and C_3.

*** Computed for Mississippi River stage 38 msl.

The elevation of the water B in the wells being pumped with deep-well turbine pumps was obtained by direct sounding; the hydrostatic head A immediately adjacent to the periphery of the gravel filter for the well was taken to be the elevation of the water in the well B plus the head loss in the well H_w (see Fig. 1) or $A = B + H_w$, which is valid for all tests reported herein as the flow to the wells was artesian. The water elevation in the wells being pumped by the centrifugal pumps was taken to be elevation of pump minus pump vacuum plus head loss in collector pipe, or $B = C - V + H_c$.

The water elevation at the periphery of the wells being pumped with the centrifugal pumps was taken to be $A = C - V + H_w = B + H_w$. The average elevation of water in the various groups of wells and the average hydrostatic head at the periphery of the wells, as well as the average net head on the wells, are given in table 3. The average net head H on the well system being tested was taken to be $H = R - A$, where R = the Mississippi River stage in ft msl, A = average elevation of the hydrostatic head at the periphery of the wells.

The average readings for various groups of piezometers considered to be representative of certain areas in the bottom of the excavation are given in table 3 together with the maximum (net) piezometric head as defined in Fig. 1 and the maximum (net) piezometric head in percent of the total net head on the well system. The average well flow for the wells being tested per foot of average drawdown and per foot of net head on the wells is given at the bottom of table 3. The average total drawdown $H_a = R$ minus average elevation of water table at periphery of wells and as measured by piezometers within the excavation. (Head losses in the wells and the collector pipes shown in Fig. 1 with two 12-in. pumps operating at each collector pipe are plotted for various well flows in Fig. 4. During the pumping tests only one pump was pumping from each collector pipe.)

Drawdown Curves

Drawdown curves as measured by piezometers on the center line of the approach and outlet channels for pumping tests C_1, C_2, D and E are shown in Fig. 5. In the vicinity of the lines of wells the hydrostatic head at the wells and the maximum head midway between wells are both plotted. From Fig. 5 it would appear that the radius of influence for the well system extends outward approximately 2000 ft. Although the near bank of the Mississippi River is only about 2200 ft from the center line of the structure and there is no known source of seepage landward of the excavation, it appears that the shape of the drawdown curves is about the same both riverward and landward of the excavation.

Hydrostatic Head, Well Flow, and Coefficient of Permeability

The net build-up in piezometric head as measured by piezometers midway between the lines of the wells (piezometer group I) was approximately 15.7 percent for test C_1. This build-up in pressure ranged from approximately 10 to 20 percent for tests C_1, C_2, C_3, and D.

The well flow ranged from about 180 to 190 gpm per foot of (average) drawdown, and about 170 to 180 gpm per foot of net head on the wells for the C and D tests. The maximum average flow per well, based on the results of test C_1, would be about 585 gpm for a river stage of 60. The flow required for adequate pressure relief for all parts of the excavation except the riprap

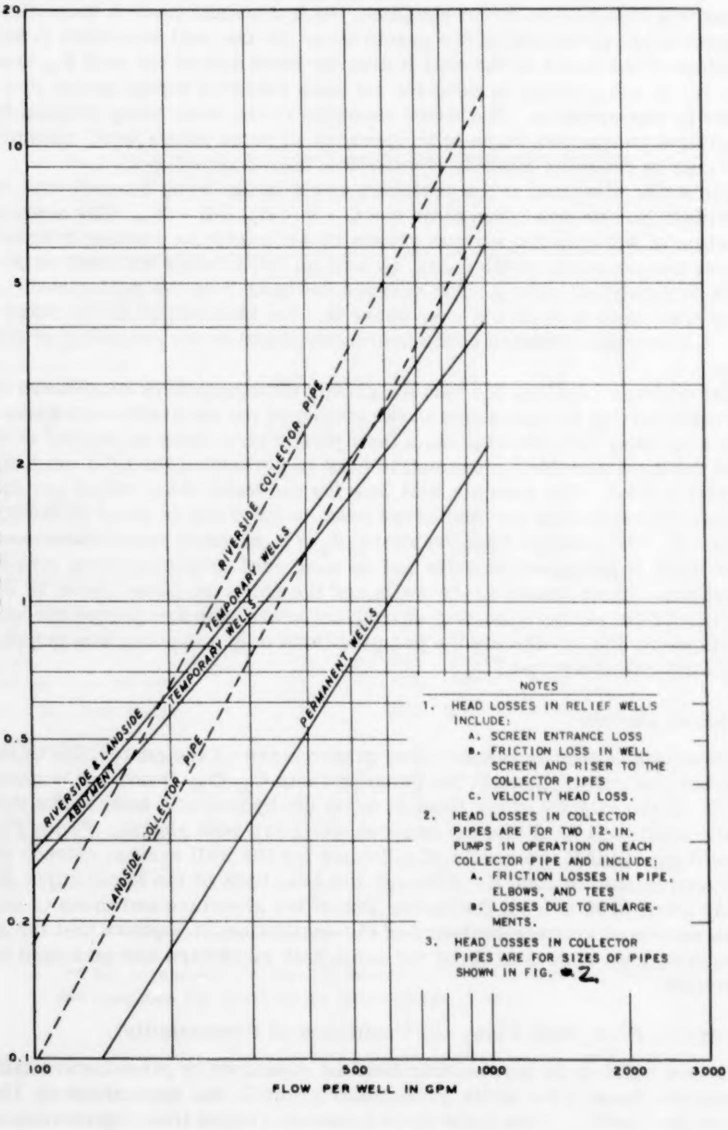

Fig. 4. Hydraulic head losses in relief wells and collector pipes

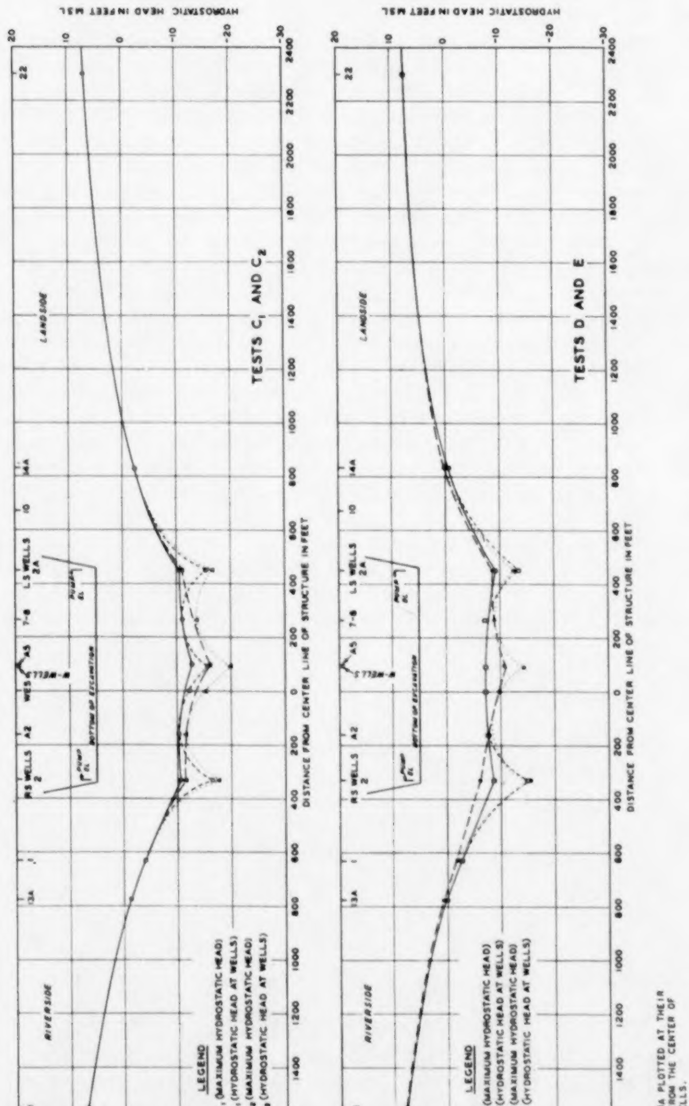

Fig. 5. Hydrostatic head in deep sands parallel to axis of channel, tests C_1, C_2, D, and E

extensions for well system C is on the average about 600 gpm per well for a river stage of 60.

During the design of the structure the flow from a dewatering system required to reduce the hydrostatic head in the deep sand was computed assuming the system of wells in the excavation the equivalent of a large-diameter well (well radius = 450 ft) in an artesian aquifer having a permeability of 1000×10^{-4} cm per sec as determined from field pumping tests and a thickness of 40 ft, and assuming a line source of seepage at the Mississippi River. The computed flow per ft of drawdown was about 170 gpm corresponding to a total required flow of about 13,000 gpm with the piezometric head in the deep sands lowered 5 ft below the bottom of the excavation and the Mississippi River at a stage of el 60 (the maximum river stage anticipated during construction). The computed value of 170 gpm per foot of average drawdown compares favorably with the observed values shown at the bottom of table 3.

The permeability k_f of the deep sand stratum (based on a thickness of 40 ft) was computed by several methods from the piezometric data and total well flow for tests C_1 and D. In method A, k_f was computed from the piezometric profile determined from piezometers 1, 13A, and 21 riverward of the excavation and piezometers 22 and 14A landward of the excavation. In method B, k_f was computed from readings of piezometers 21 and 22 and the average head beneath the excavation, assuming the well system in the excavation equivalent to a large-diameter well with a 450-ft radius. In method C, k_f was computed on the basis of the average head beneath the excavation, an infinite line source of seepage 2200 ft from the center of the excavation, and assuming the well system in the excavation equivalent to a large-diameter well with r_w = 450 ft. The results of the above computations are summarized below.

Pumping Test	Piezometer Line	Coefficient of Permeability of Deep Sand Stratum in 10^{-4} cm per sec Units by Various Methods: A	B	C
C_1	Riverside	1235	1100	1225
	Landside	1875	1290	- -
	Average	1235*	1195	1225
D	Riverside	1190	1170	1260
	Landside	1890	1485	- -
	Average	1190*	1330	1260

Average of methods A, B, and C = 1240×10^{-4} cm per sec.

*Does not include value computed for landside piezometer line.

The average permeability of 1240×10^{-4} cm per sec is somewhat greater than the value of 1040×10^{-4} cm per sec obtained from the previously referenced field permeability pumping tests on a single well. However, the aquifer thickness in the vicinity of the single well is only about 40 ft, whereas the average thickness of the deep sand aquifer beneath the entire excavation is somewhat greater than 40 ft. Therefore, the permeability computed from tests C_1 and D on the basis of an aquifer thickness of 40 ft is probably somewhat greater than the actual over-all permeability for the aquifer. In view of this it is considered that the over-all permeability of the deep sand aquifer is

between about 1050 and 1250 x 10^{-4} cm per sec.

Allowable Hydrostatic Head and That Obtainable with Existing Well System

The specifications for construction of the low sill structure require that the hydrostatic head in the deep sands be kept 5 ft below the bottom of the excavation when the excavation is at final grade. The elevation of the bottom of the excavation at key points is shown in Fig. 2. Allowable heads at selected points in the excavation are shown in table 6.

The hydrostatic head in the deep sands that can be obtained with the existing well system and various combinations of wells was computed for Mississippi River stages up to el 60 from observed test data for well systems C_1, C_2, D, and E, and theoretically for well system F. The results of the computations for a river stage of 60 ft msl are given in table 5. (Well combination C_3 was not analyzed, as preliminary computations indicated that the flows from the permanent wells would be excessive for this case with the river at el 60.) The procedure for estimating the obtainable hydrostatic head with a given combination of wells consisted of extrapolating the piezometric and well flow data obtained during the pumping tests on that combination of wells as described below. The net hydrostatic head h for a given piezometer or group of piezometers in per cent of the net head H on the well system was determined from the pumping tests. As the net head H on the well system depends upon the head losses in the wells and collector systems, it was necessary to compute the well flows and corresponding head losses with the river at different stages and then determine H and h. The flows from the lines of riverside and landside temporary wells were determined as follows. With the excavation to final grade, the centrifugal pumps for the riverside and landside lines of temporary wells will be set at el -10 and the vacuum at the pumps will be about 25 ft. The average bottom of the excavation will be about el -15, or 5 ft below the elevation of the pumps. Thus with a 25-ft vacuum at the pump, the maximum possible drawdown in the riverside and landside lines of temporary wells would be 20 ft below the bottom of the excavation. The flow from each of these two lines of wells was computed for different river stages, assuming a frictionless well and collector system, from values of Q/H obtained from the pumping test multiplied by the head H for a frictionless well system. These values of Q were then plotted versus corresponding drawdowns of elevation head losses (e.g., see line H_E, Fig. 6). The flow that can be pumped from the actual well system, taking into consideration the effect of hydraulic head losses in the wells and the collector system for various drawdowns in the wells below el -15, was obtained by adding the head loss in the wells H_w and in the collector pipe H_c to the effective drawdown H_E below the bottom of the excavation (or elevation head) for the frictionless system at various assumed well flows, as illustrated in Fig. 6. The maximum flow from the well system is that where the Q vs $(H_w + H_c + H_E)$ line equals the maximum available drawdown below the bottom of the excavation (= 20 ft). The hydraulic head losses in the wells H_w and collector pipe H_c are those corresponding to the actual attainable flow from the system (see Fig. 6). The average of $H_w + H_c$ for both the riverside and landside lines of temporary wells was added to the drawdown elevation (-35) obtainable with a frictionless system to obtain the elevation of the average head A at the periphery of the wells.

The net head on the well system H was taken as the difference between the

Table 5

Estimated Well Flows and Hydrostatic Heads Beneath Excavation for Various Combinations of Wells for a Mississippi River Stage of 60 msl

Well Combination	Hydrostatic Head or Elevation C_1	C_{1-a}*	C_2	D	E	F
Avg hydrostatic head at periphery of wells, msl	-29.4	-29.9	-30.4	-27.5	-26.5	-
Avg net head on wells, ft	89.4	-89.9	90.4	87.5	86.5	-
Avg max (net) piezometric head, msl:						
Piezometer group I	-15.4	-18.3[d]	-20.6	-9.5	-10.0	-0.4[a]
Piezometer group II	--	--	--	--	-5.6	--
Piezometer 13-A	8.8	8.1	7.6[c]	10.2	13.2	13.2[b]
Avg for excavation	-15.4	-18.3	-20.6	-9.5	-10.0	--
	Well Flow in gpm					
Riverside wells (total)	4,380	4,080	3,780	5,060	2,990	0
Avg flow per well	485	455	420	565	995	0
Landside wells	4,330	4,115	3,900	5,250	2,970	0
Avg flow per well	620	590	555	750	990	0
Abutment wells	3,650	3,525	3,400	4,510	3,840	2,400
Avg flow per well	910	880	850	1,125	960	600
Permanent wells	3,400	4,343	5,790	0	3,580	4,200
Avg flow per well	485	620	825	0	511	600
Total well flow	15,760	16,063	16,370	14,820	13,380	6,600
Avg flow per well	585	595	605	740	780	600

a - Computed at piezometer A-2 and based on an assumed well flow of 600 gpm for the permanent and abutment wells with the river at el 38 msl.

b - Hydrostatic head 630 ft riverward of center line of structure on center line of channel with river at el 38.

c - Estimated from readings of piezometer 13-A for tests C_1 and C_3.

d - Average of piezometers A-1, A-3, 7, and 8.

* Computed from results of tests C_1 and C_2.

Table 6

Hydrostatic Head Allowable with Excavation to Grade and Attainable with Deep Dewatering System with Centrifugal Pumps Set at El -10 and Mississippi River Stage = 60 msl

Area	Allowable Hydrostatic Head: Shallow Bay Excavation	Allowable Hydrostatic Head: Deep Bay Excavation	Attainable* Hydrostatic Head Well System: C_1	Attainable* Hydrostatic Head Well System: C_{1-a}**
Approach: Piezometers A-1, A-2, A-3	-13	-18	-15.5	-16.6
Center line structure: WES well	-13	-19	-20.2	-23.6
Stilling basin: Piezometer A-5	-18	-25	-21.8	-25.4
Outlet channel: Piezometers 7 and 8	-13	-18	-16.9	-19.8
Riprap extensions: Piezometer 13-A	-13	-	8.7	8.1

* Vacuum at centrifugal pumps = 25 ft. Collector pipes for riverside and landside temporary wells as shown in fig. 2.

** Based on a maximum flow of about 600 gpm from permanent wells and about 900 gpm for the temporary abutment wells. Values computed by averaging results computed from tests on well systems C_1 and C_2.

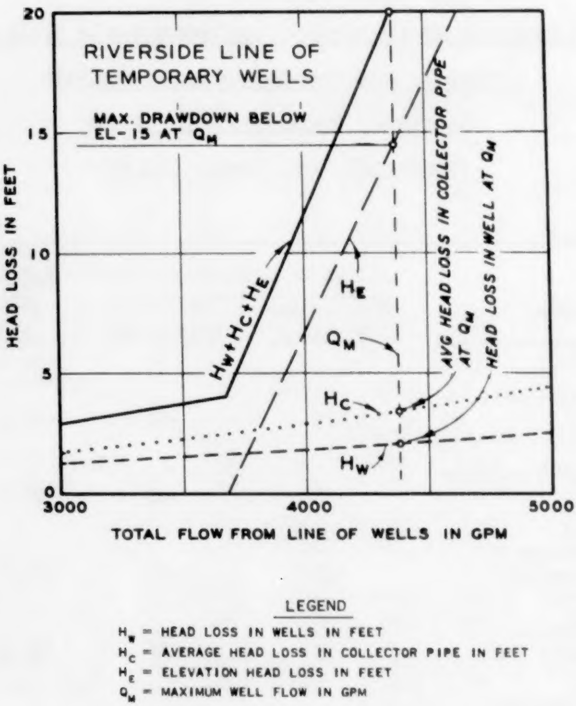

Fig. 6. Determination of maximum flow from temporary wells with centrifugal pump, well system C_1

river stage and el A. Using this value of H the head h at the piezometers was computed assuming the same value of h/H as that observed during the pumping tests on the corresponding well system. The head at the piezometers was then converted into piezometer readings in msl by adding the head h to el A. A plot of the hydrostatic head for selected piezometer groups and various combinations of wells versus the corresponding Mississippi River stage with the centrifugal pumps set at el -10 and a pump vacuum = 25 ft is shown in Fig. 7.

The flows from the 4 temporary wells at the abutments and the 7 permanent wells were estimated from the total net head on the wells and the relationship between flow per ft net head observed in the pumping tests on the corresponding well systems. The corresponding total flows from the relief wells and average flow per well for a river stage of 60 ft msl are shown in table 5.

From table 6 it appears that well system C_1 is not capable of reducing the hydrostatic head beneath the deep bay section of the excavation to that required by the specifications, but that the hydrostatic heads could be reduced satisfactorily by increasing the pumping rate from the permanent wells (case C_2). However, the average flow from the permanent wells for case C_2 is about 825 gpm, which is considered excessive, inasmuch as the wells were tested for sand infiltration at a flow of only 500 gpm. In view of this it was considered that a pumping rate for the permanent wells intermediate between C_1 and C_2, herein termed C_{1-a}, might accomplish the required pressure relief without the flows from the permanent wells exceeding 600 gpm, a rate of flow considered satisfactory. The estimated flow from the relief wells and the hydrostatic pressure at the piezometers for case C_{1-a} were obtained by interpolating between the estimated flows and hydrostatic heads for cases C_1 and C_2. The heads and well flows for case C_{1-a} are shown in table 5 for a Mississippi River stage of 60 ft msl, and the hydrostatic head at selected piezometers is plotted versus river stage in Fig. 8.

The hydrostatic head at selected piezometers while pumping on the 4 temporary abutment wells and the 7 permanent wells (well system F) were computed by means of image wells, assuming a flow of 600 gpm per well, a line source of seepage 2000 ft from the center of the excavation (about at the high bank of the Mississippi River), and a 40-ft-thick sand stratum with a permeability of 1100 x 10^{-4} cm per sec. The hydrostatic head for piezometer group I and piezometer 13A is shown in table 5 and Fig. 7. It was found that with the excavation to grade the flows from the permanent and temporary abutment wells will exceed 600 gpm for river stages above 38 msl.

The total well flow for the different well combinations and cases is shown for various river stages and the excavation to grade in Fig. 9. It appears from this figure that a maximum flow of about 16,000 gpm (case C_{1-a}) would have to be pumped for a river stage of 60.

Adequacy of System

Well System C

Based on the hydrostatic heads in table 5 and a comparison between the allowable and attainable hydrostatic heads in the deep sand in table 6, it appears that adequate pressure reduction can generally be achieved by well

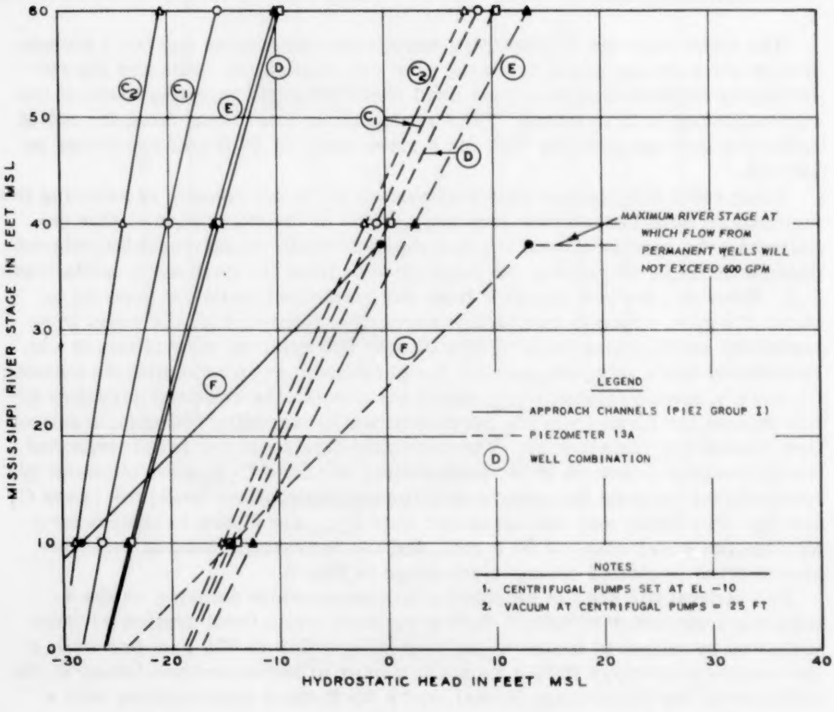

Fig. 7. Hydrostatic head in deep sand beneath excavation for various river stages and combinations of wells

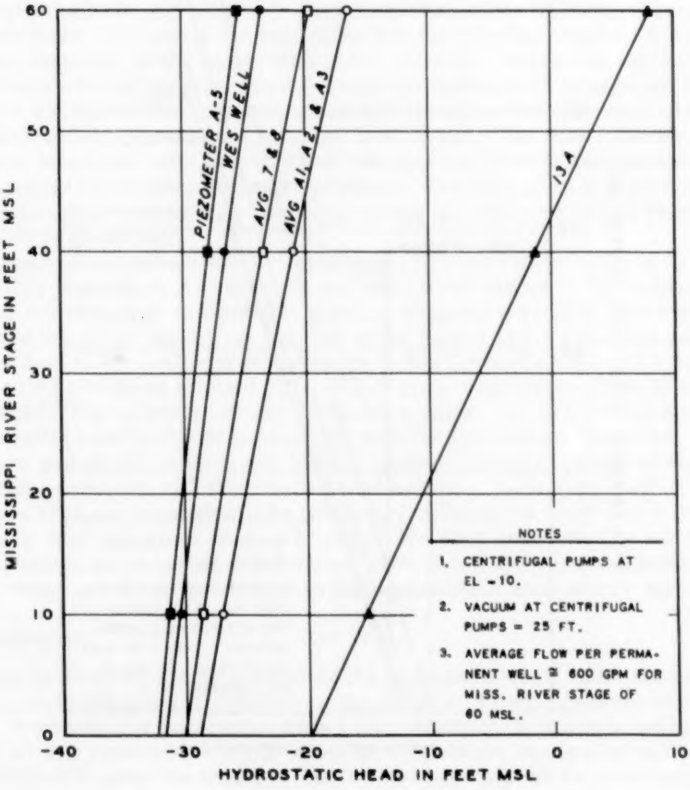

Fig. 8. Hydrostatic head in deep sand beneath excavation for well system C_{1-a}

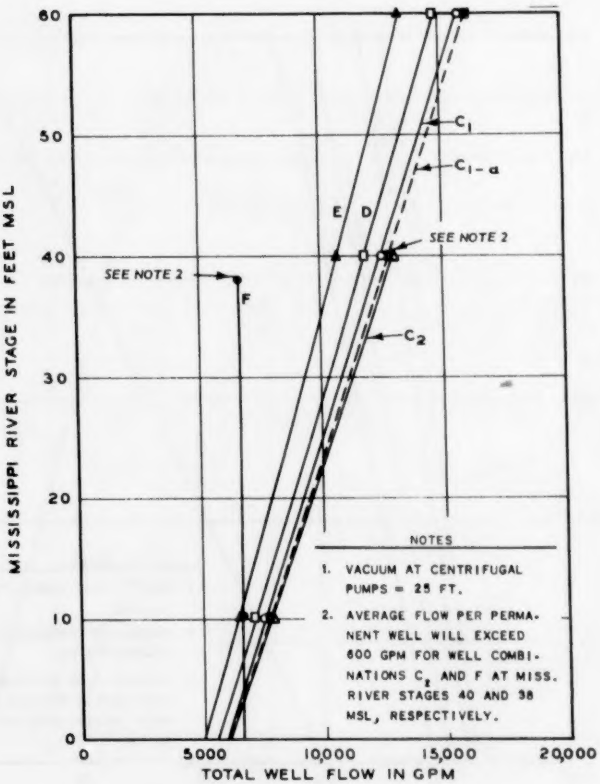

Fig. 9. Total well flow for various river stages and combinations of wells for centrifugal pumps at el -10

system C with a river stage of 60 by pumping the riverside and landside temporary wells at a vacuum of 25 ft with the pumps set at el -10, the permanent wells at a rate of about 600 gpm, and the abutment wells at about 880 gpm. However, well system C is not adequate for reducing substratum pressures to satisfactory values in the riprap extensions (vicinity of piezometers 13A and 14A) and in the excavation for the approach channel for the deep bays (adjacent to the center line of the channel). In general, except for the approach channel and riprap extensions, the hydrostatic head would be about 0 to 4 ft below that required by the specifications with the excavation to final grade and the Mississippi River at el 60 ft msl. The flow from the temporary abutment wells will be about 880 gpm per well, which exceeds the rated capacity of the deep well turbine pumps. However, this flow probably can be obtained by speeding up the pumps in the abutment wells if a river stage of 60 is predicted.

The hydrostatic pressure in the approach channels can be reduced to satisfactory amounts by reducing the hydraulic head losses in the collector pipes for the riverside and landside lines of temporary wells by increasing the size of the pipes about 2 in. and that at the pumps about 4 in. The average head loss in the collector pipes shown in Fig 2 would be about 3.2 ft, which could be reduced to about 1.0 to 1.5 ft by increasing the sizes of the pipes, which in turn would result in a corresponding 1.5- to 2.0-ft reduction in hydrostatic head in the deep sands beneath the excavation. However, it will not be possible to reduce hydrostatic heads to required values beneath the riprap extensions with the existing well system. To further reduce pressures in these areas it will be necessary to install at least one 8-in. ID temporary well with about 30 ft of screen in the deep sand near the end of each extension, or to install about six or more 3-in.-diameter wellpoints with 15-ft screens and 3-in. riser pipes in the deep sand in each riprap extension.

Well System D

It may be desirable to stop pumping the permanent wells while placing the collector pipe and pouring concrete in the vicinity of the permanent relief wells. In the shallow bay section of the excavation the temporary wells (well system D) will reduce the hydrostatic head beneath the excavation to that allowable (el -17) after the drainage blanket has been placed for river stages up to 34 msl. However, in the deep bay section the hydrostatic head will exceed that allowable (el -24) after the drainage blanket has been placed at river stages above 9 msl.

Well System E

Well system E may be used when backing out of the excavation after the drains, base slabs, impervious blankets and riprap have been placed. Under these conditions the hydrostatic head in the deep sand may be allowed to rise to an elevation 5 ft above the surface of the riprap, concrete, or tailwater as the structure excavation is flooded. Thus, with the base slabs poured and riprap placed in the bottom of the excavation and no water in the excavation, the allowable head would be about 0 msl in the approach and outlet channels and riprap extensions and 0 to -7 in the structure areas. Well system E can satisfactorily reduce the hydrostatic head beneath the structure and in the approach and outlet channels for river stages up to 60 msl; it can reduce the

head beneath the riprap extensions to el 0 for river stages up to 35 msl.

To estimate the capabilities of well system E for any river stage and the excavation flooded, the hydrostatic head at piezometer groups can be estimated from the head in per cent of total net head given in table 3 and the estimated total net head on the well system. The head at a given group of piezometers with frictionless wells and collector systems would be equal to the head in per cent given in table 3 multiplied by the net head on the well system. The actual piezometric head will be about 1 to 3 ft higher, as a result of head losses in the relief wells and collector systems.

Well System F

Well system F, in which only the 4 temporary abutment wells and 7 permanent wells are pumped at a flow of 600 gpm per well, may be used when backing out of the excavation. This system will satisfactorily reduce hydrostatic heads beneath the main part of the excavation for river stages up to about 38 msl, with no water in the excavation (net head on wells = 53 ft). At net heads greater than 53 ft, the well flows will exceed 600 gpm. However, the head beneath the riprap extensions will be excessive at river stages above el 25 prior to flooding. An estimate of the adequacy of well system F at any river stage with the structure excavation flooded can be obtained from the data given in table 5. For example, the maximum net head at piezometer group I will be -0.4 msl with the river at 38 msl, representing a drawdown of 38.4 ft. This drawdown will be constant while pumping well system F at a flow of 600 gpm per well. Thus the piezometer reading in feet msl for group I will be the river stage minus the drawdown of 38.4 ft at the piezometer for any river stage at net heads on the wells less than 53 ft. This computed reading can then be compared with that required by the specifications for the given water surface inside the excavation.

Summary and Conclusions

On the basis of the pumping tests and analyses presented in this paper it is concluded that:

a. The deep well system appears to be reducing satisfactorily the hydrostatic pressures in the deep sands underlying the excavation. Lowering of the hydrostatic head in the deep sand stratum below the bottom of the excavation has also had a material effect on drying the excavation slopes and the bottom of the excavation.
b. The 20 temporary wells and 7 permanent wells are adequate for controlling hydrostatic pressures in the deep sands beneath the main portion of the excavation for river stages up to el 60, provided the diameter of the collector pipes for the riverside and landside temporary wells is increased somewhat. It may be necessary to pump each of the permanent wells at rates up to 600 gpm and the temporary abutment wells at rates of flow up to 900 gpm for such a river stage.
c. Adequate pressure reduction in areas of riprap extensions cannot be achieved with the existing well system for river stages in excess of 14 msl. For adequate pressure reduction in these areas at least one 8-in. ID temporary well with about 30 ft of screen in the deep sand should be installed near the end of each extension, or about 6 or more 3-in.-diameter wellpoints with 15-ft screens and 3-in. riser pipes should be

installed in each riprap extension.

d. A well flow of about 16,000 gpm is indicated for a river stage of 60 and the excavation to final grade. This corresponds to an average flow of about 600 gpm per well.

e. The well flow per foot of net head on the well system is about 170 to 180 gpm, and about 180 to 190 gpm per foot of average drawdown.

f. The observed flow per foot of average drawdown compared favorably with the flow per foot drawdown computed assuming the system of wells equivalent to a single large-diameter artesian well driven by a line source (the Mississippi River).

g. The average permeability of the deep sand aquifer as computed from well tests C_1 and D was about 1050 to 1250 x 10^{-4} cm per sec as compared to 1040 x 10^{-4} cm per sec determined from a previous field pumping test.

ACKNOWLEDGMENT

The dewatering system was installed and pumped by the Independent Wellpoint Corporation. The pumping tests on the system of deep wells were performed by representatives of the Independent Wellpoint Corporation, New Orleans District, and the Waterways Experiment Station. The authors actively participated in the studies and in the analyses of the data and were assisted in the analyses by Mr. J. D. Perrine, J.M., ASCE, of the Waterways Experiment Station.

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Appendix A

The piezometric and well flow data obtained from pumping tests C - E are plotted on plans contained in this appendix showing the dewatering system, excavation, and piezometers to facilitate visualizing the relationships between well discharges and piezometer observations made during the various pumping tests. The above data are shown in Figs. A-1 through A-5, inclusive, which follow.

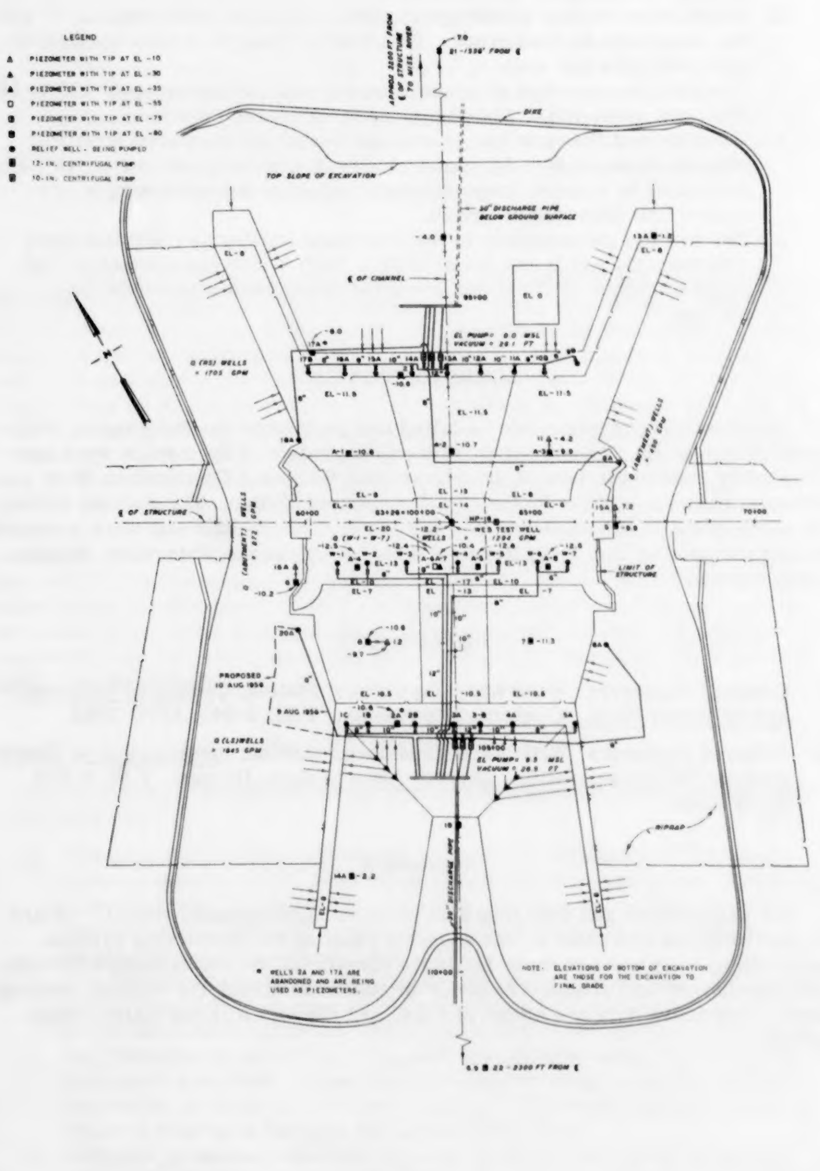

Fig. A-1. Well flow and piezometric data for pumping test C_1

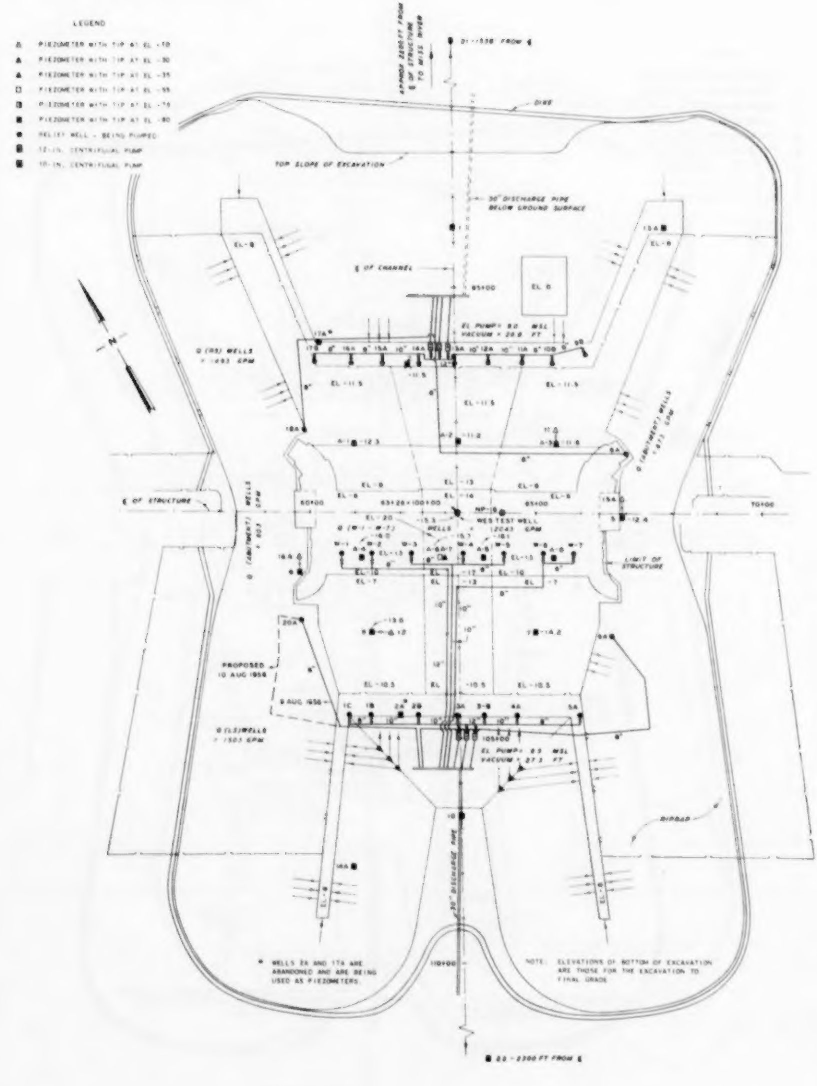

Fig. A-2. Well flow and piezometric data for pumping test C_2

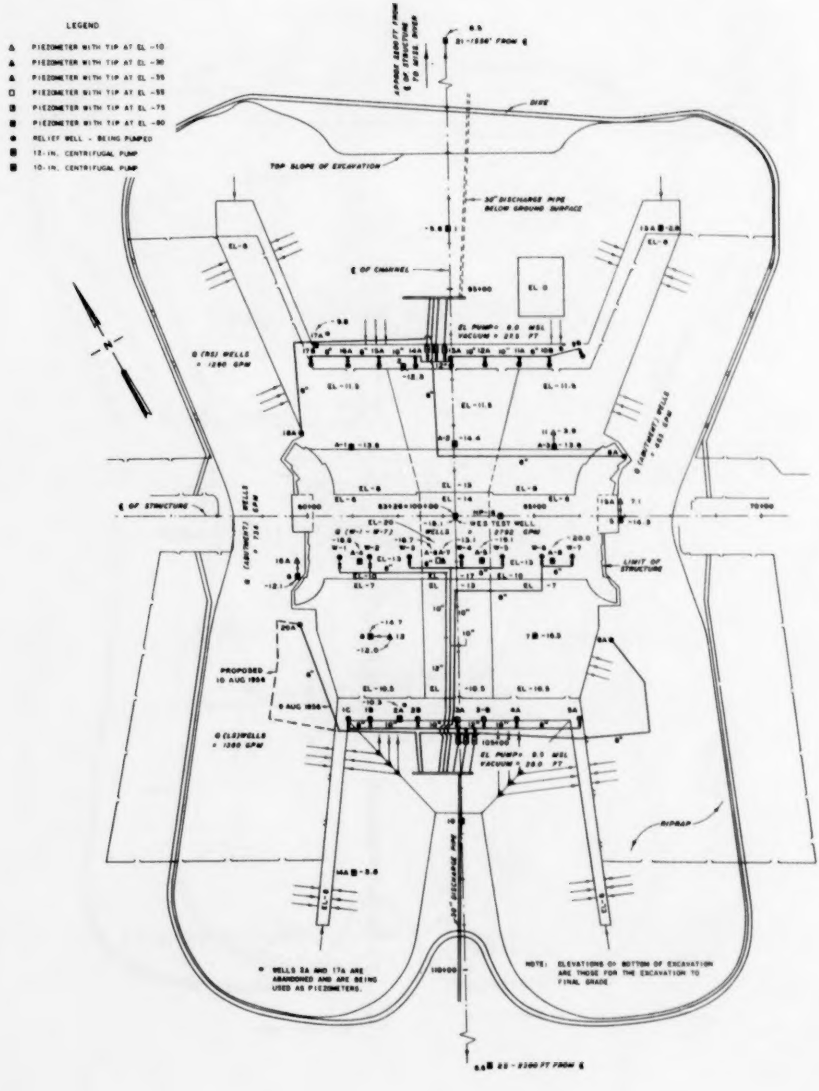

Fig. A-3. Well flow and piezometric data for pumping test C_3

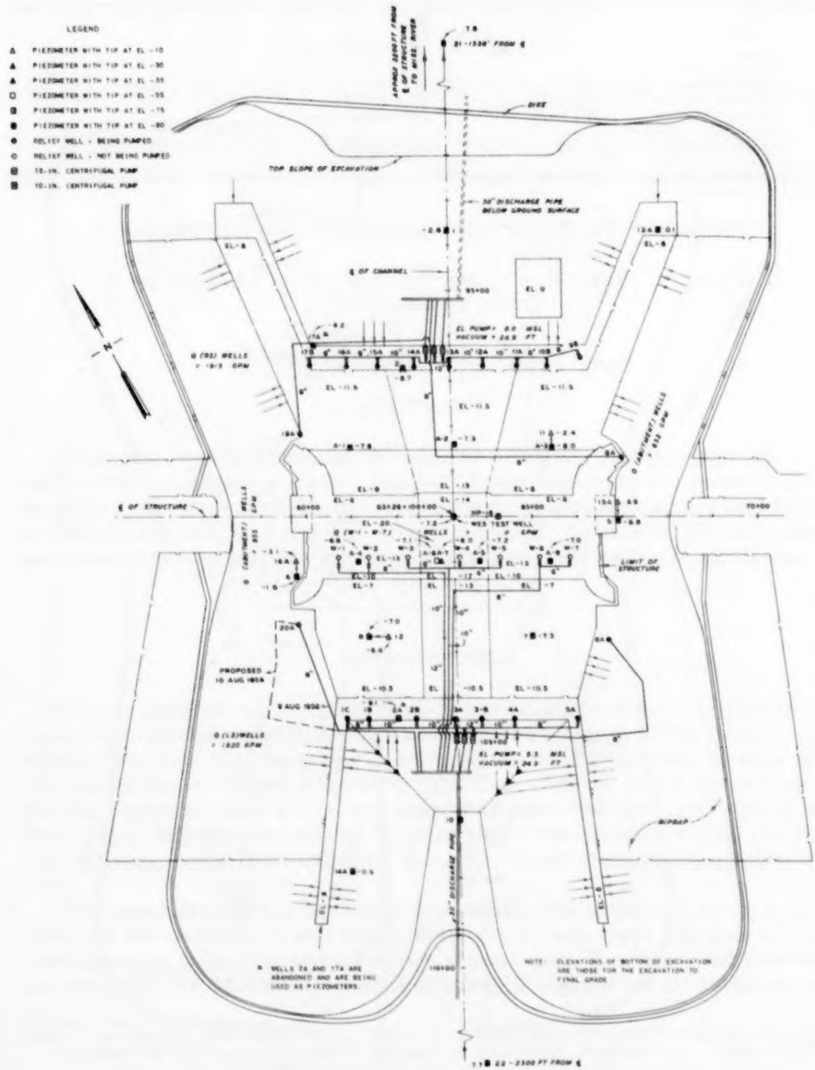

Fig. A-4. Well flow and piezometric data for pumping test D

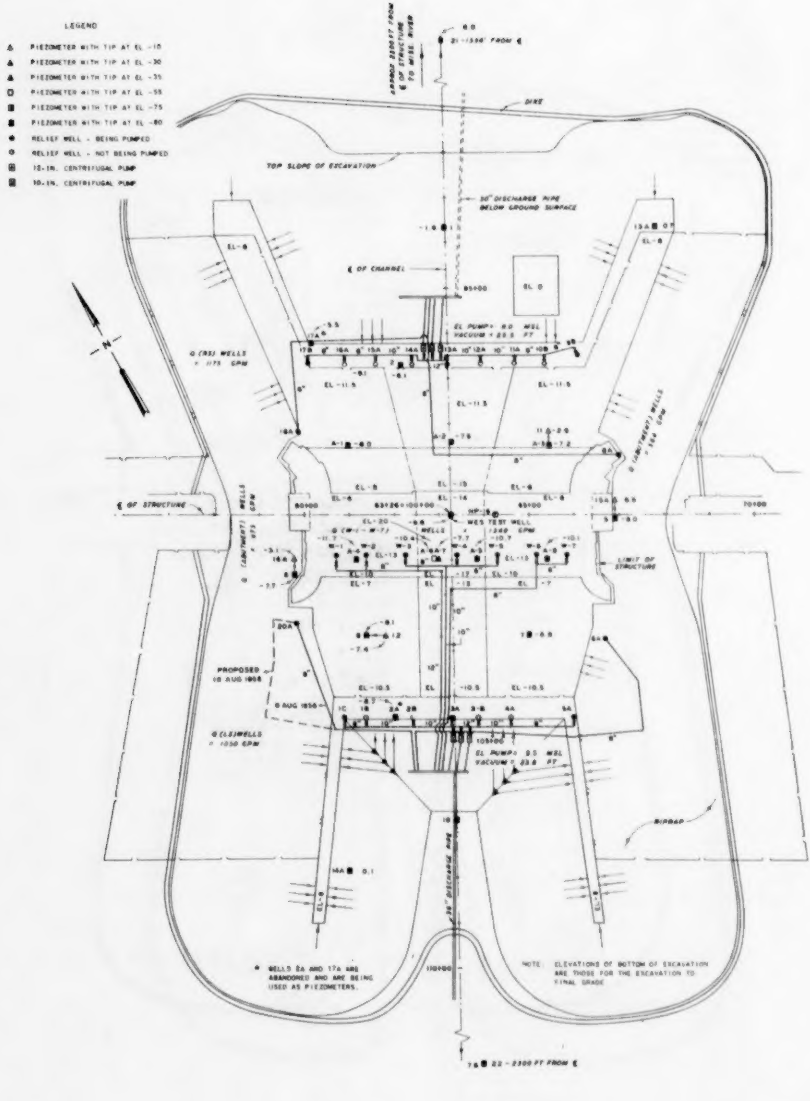

Fig. A-5. Well flow and piezometric data for pumping test E

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A METHOD TO DESCRIBE SOIL TEMPERATURE VARIATION

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(Proc. Paper 1537)

ABSTRACT

From observed data, values for thermal diffusivity, temperature and temperature amplitude were determined. These constants were used in an equation to describe the variation of temperature with time at any soil depth for both Lexington, Kentucky and Ottawa, Ontario. Calculated temperatures and mean observed soil and air temperatures are plotted for comparison.

INTRODUCTION

In many types of installations, such as buried high voltage transmission lines, underground water lines, and underground portions of buildings and other structures, it is desirable to know the thermal properties of soils at various locations. These properties vary from place to place, due to average moisture content, type of soil, and amount of solar radiation, and vary at any given place with depth of soil and time of year. The literature does not provide the engineer with a method by which he can approximate these properties.

This paper proposes a method of determining the necessary properties to describe the variation of soil temperature due to soil depth and time of year, and compares calculated temperatures at Lexington, Kentucky, and Ottawa, Ontario, with the observed soil temperatures, the observed air temperatures,

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the amount of solar radiation, and the amount of rainfall. This method requires observed soil temperatures at a minimum of two depths for one year.

Method

Consider a semi-infinite homogeneous solid bounded only by the yz plane at x = 0. For transient heat flow in this case, Eq. (1) applies provided the temperature of this yz plane, or surface follows a sine wave variation.

$$t = t_m + t_a e^{-x\sqrt{\frac{\pi}{\alpha P}}} \sin\left[\frac{2\pi\tau}{P} - x\sqrt{\frac{\pi}{\alpha P}}\right] \qquad (1)$$

Where:

- t = calculated temperature at any point, $^{\circ}$F.
- t_m = the mean temperature about which this sine wave oscillate, $^{\circ}$F.
- t_a = surface amplitude of variation of this sine wave, $^{\circ}$F.
- x = depth below surface, ft.
- α = thermal diffusivity, ft^2/hr.
- P = period, 1 year, 12 months or 8766 hours.
- τ = time, months.
- e = Naperian base.

This equation can be satisfactorily applied to the ground since in most locations the monthly averages of the surface temperature follow roughly a sine wave variation, having a period of one year. The deeper under the surface the temperature is measured, the more closely it approaches a sine wave. This equation has previously been used in soil calculations,[1,3] in conjunction with a heat pump ground coil study.

The real problem involved is that of determining the constants for this equation. The average value of the temperature at a given depth is calculated from

$$t \text{ average} = \frac{t_{Jan.} + t_{Feb.} + \cdots\cdots t_{Dec.}}{12} \qquad (2)$$

where the temperatures in the numerator are the monthly average temperatures for that depth. The yearly mean value of t_m used in Eq. (1) is taken as the mean of the t average at the different depths or planes.

Or:

$$t_m = \frac{\begin{matrix} t \text{ average} \\ \text{at } x = x_a \end{matrix} + \begin{matrix} t \text{ average} \\ \text{at } x = x_b \end{matrix} \cdots\cdots}{N} \qquad (3)$$

where N is the number of terms in the numerator. Since the value of t average for planes on or near the surface differs more than one half degree from those at greater depths, it is felt that the overall accuracy of this method is increased by using only values of t_m calculated from t average values at depths greater than 2 feet.

The next constant in Eq. (1), t_a, represents the maximum surface variation or amplitude about the mean temperature. Since the observed surface temperature is subjected to every whim of nature, it is felt that the value of t_a should be based on data excluding temperatures near the surface. The term $t_a e^{-x_a\sqrt{\frac{\pi}{\alpha P}}}$ represents the maximum temperature amplitude about the mean at a depth of $x = x_a$ ft. or

$$t_{x_a} = t_a e^{-x_a\sqrt{\frac{\pi}{\alpha P}}} \tag{4}$$

The amplitude at any given plane, x_a, is calculated by

$$t_{x_a} = \frac{t \text{ max. at } x_a - t \text{ min. at } x_a}{2} \tag{5}$$

The ratio of the observed temperature amplitudes of any two planes is given by

$$\frac{t_{x_a}}{t_{x_b}} = e^{-(x_a - x_b)\sqrt{\frac{\pi}{\alpha P}}} \tag{6}$$

or

$$\frac{\log_e \frac{t_{x_a}}{t_{x_b}}}{x_b - x_a} = \sqrt{\frac{\pi}{\alpha P}} = r, \text{ a constant} \tag{7}$$

Excluding data taken at any depth less than two feet, the value of the constant r is determined between all successive planes. A mean value of r (r_m) is calculated by

$$r_m = \frac{r_1 + r_2 + \text{-----}}{N} \tag{8}$$

where N again is the number of terms in the numerator.

Once the value of r_m is known, the difference in the amplitudes between successive planes may be calculated. The observed amplitude at the deepest plane is the value most likely to remain fixed. From this value, the temperature amplitudes at all other planes can be calculated, including that of the surface, t_a. Now since

$$r_m = \sqrt{\frac{\pi}{\alpha P}} \tag{9}$$

the average value of α required to give this constant may be calculated.

If the date when $\tau = 0$ is defined as April 15, then the sine term in Eq. (1) will have to be changed to read

$$\sin\left[\frac{2\pi}{P}(\tau-\tau_0) - x\sqrt{\frac{\pi}{\alpha P}}\right] \tag{10}$$

where τ_0 is the time interval between April 15 and the date that the surface temperature crosses the mean temperature. The term $\Delta\tau$, defined as:

$$\Delta\tau = \Delta x\sqrt{\frac{\pi}{\alpha P}} \tag{11}$$

represents the time-lag between planes, which can be determined from the observed data. This allows another independent calculation of α.

After trying many methods of determining values of time-lag and α from the observed data, it has been found that the most accurate method that requires no previous experience starts with determining the date when each curve reaches a maximum. This is defined very simply as the mid-point between the two points where the observed temperature curve crosses the mean. Assuming each month is exactly one-twelfth of a year, and that the observed monthly average actually exists on the 15th., the curves will cross the mean exactly three months before the observed maximum, or in most cases, some time after April 15.

The time in months between the times any two curves cross the mean represents the time lag between those planes, and when divided by the difference in depths of the planes, gives the time lag per foot, expressed as $\Delta\tau/\Delta x$. This value is determined between the deepest plane and each other plane except the surface. The average of these values gives the time lag per foot, which Eq. (1) requires to be constant. The total time lag between the deepest plane and the surface plane is now calculated from the above average. Subtracting this total time from the calculated date that the deepest curve crosses the mean gives the date the calculated surface curve crosses the mean. τ_0 is the time after April 15 that the surface curve crosses the mean. If this happens before April 15, it simply means that τ_0 is negative.

The value of α may now be checked from the time-lag by the following equation.

$$\alpha = \left(\frac{12}{2}\right)^2\left(\frac{\Delta x}{\Delta\tau}\right)^2\frac{\pi}{P} = \frac{36}{\pi P\left(\frac{\Delta\tau}{\Delta x}\right)^2} \tag{12}$$

In most cases the two values of α will not agree, but since they both have been calculated, it seems better to use both than to use an average value. Then

Eq. (1) can be changed to show two values of α, which is the final form used in subsequent calculations.

$$t = t_m + t_a e^{-x\sqrt{\frac{\pi}{\alpha_1 P}}} \sin\left[\frac{2\pi}{P}(\tau - \tau_0) - x\sqrt{\frac{\pi}{\alpha_2 P}}\right] \tag{13}$$

where

α_1 = difusivity determined from amplitude

α_2 = difusivity determined from time-lag

Data and Results for Lexington, Kentucky

Measurement of Soil Temperatures

Soil temperatures were recorded from the surface[4] of the earth down to a depth of 10 ft., at two foot intervals by the use of an L. & N. Speedomax recorder. The average daily soil temperature for each depth was obtained from readings taken at two-hour intervals throughout each day and night. The monthly average soil temperatures were calculated from the daily average temperatures, and are listed in Table 1 for the year 1952.

Measurement of Solar Energy

A 50-junction type phrheliometer (Fig. 1) was mounted on the top of Anderson Hall at the University of Kentucky for the purpose of measuring the intensity of Solar radiation which falls on a horizontal plane(3,4). The solar energy was recorded by the use of an L. & N. Micromax strip chart recorder (Fig. 2). The solar energy received by a horizontal surface for a day was obtained by measuring the area under a curve traced on the chart (Fig. 3). The average solar energy measured at Lexington, Kentucky, for 1952 is listed in Table 1.

Rainfall and Air Temperature

The mean air temperature and rainfall for 1952, listed in Table 1, were observed at the Blue Grass Airport which is about 5 miles distant from Anderson Hall. These data were supplied by the United States Weather Bureau.

Type of Soil at Lexington

The soil at Lexington has a General Casagrande classification of lean clay. A representative sample from the profile where thermocouples were buried, had a liquid limit of 43.4%, a plasticity index of 16.1%, and a unit weight, dry, of 95 lbs/ft^3.

Equation for Lexington

From the data listed in Table 1, the following constants, required in Eq.

4. Here the surface temperature is defined as the temperature recorded by a thermocouple junction buried at depth of one-half an inch in the soil.

(13), were calculated.[5]

$$t_m = 58.15^\circ \text{ F.}$$
$$t_a = 20.97^\circ \text{ F.}$$
$$\tau_o = 0.21 \text{ months} = 6.3^\circ$$
$$\alpha_1 = 0.0291 \text{ ft}^2/\text{Hr.}$$
$$\alpha_2 = 0.0261 \text{ ft}^2/\text{Hr.}$$

For Lexington, Eq. (13) becomes:

$$t = 58.15 + 20.97e^{-0.111x} \sin(30\tau - 6.3 - 6.7x) \tag{14}$$

Eq. (14) was used to calculate soil temperature at Lexington at the 15th of each month for depths of 0, 2, 4, 6, 8 and 10 ft. The results are listed, together with the average observed temperatures, in Table 2, and are shown graphically in Fig. 4.

Data and Results for Ottawa, Ontario

Soil Temperature Data

Soil temperatures at Ottawa were taken at mid-day during 1950. The data were shown graphically by Legget and Crawford of National Research Council of Canada, in a paper presented on May 26, 1952 at the American Water Works Association Section Meeting in Montreal.(4) The curves shown in their paper represent average temperatures during two week periods. Soil temperatures listed in Table 3 were read from these curves.[6]

Solar Energy, Rainfall and Air Temperature

The solar energy, rainfall and air temperatures for 1950 were taken at the Uplands Airport. These data are shown in Table 3.[7]

Type of Soil at Ottawa

Soil classification was not available for the Ottawa soil.

Equation for Ottawa

From the data listed in Table 3, the following constants were calculated:

$$t_m = 46.91^\circ \text{ F.}$$
$$t_a = 21.72^\circ \text{ F.}$$
$$\tau_o = 0.17 \text{ months} = 51.1^\circ$$
$$\alpha_1 = 0.0128 \text{ ft}^2/\text{hr.}$$
$$\alpha_2 = 0.0129 \text{ ft}^2/\text{hr.}$$

5. See Appendix for sample calculations.
6. These data were used with the permission of the authors.
7. Courtesy of Mr. Andrew Thomson, Department of Transport, Meteorological Division, Toronto, Ontario.

For Ottawa, Eq. (13) becomes:

$$t = 46.91 + 21.72 e^{-0.171X} \sin(30T - 5.1 - 9.6X) \tag{15}$$

Soil temperatures at Ottawa were calculated for depths of 1, 2, 4 and 8 ft. The results are listed in Table 4, together with average observed temperatures, and are shown graphically in Fig. 5. The average solar energy and the rainfall are also presented in Fig. 5.

CONCLUSION

The variations of soil temperatures with time and depth at Lexington and Ottawa are satisfactorily described by Eq. (14) and (15) respectively.

$$t = 58.15 + 20.97 e^{-0.111X} \sin(30T - 6.3 - 6.7X) \tag{14}$$

$$t = 46.91 + 21.72 e^{-0.171X} \sin(30T - 5.1 - 9.6X) \tag{15}$$

In Fig. 4, the calculated surface temperature for Lexington shows a maximum error of about 9^0 F. from the observed monthly average temperature at one point, but since the observed temperature actually varies as much as 20^0 F. each 24 hours, this calculated temperature is certainly acceptable until some better method is presented. The maximum error at 10 ft. is only 1.31^0 F. and only one other calculated value at this depth is in error more than 0.5^0 F.

The difference between the calculated values of α_1 equal to 0.0291 and α_2 equal to 0.0261 is not nearly as significant as it might appear, since the maximum surface temperature calculated by Eq. (14) will change less than 2 degrees if the value of α is doubled. This indicates that these values are in rather close agreement, and that they verify a value of 0.019 determined in 1949 by an entirely different method.(5)

The depth at which the temperature variation becomes insignificant can be easily determined as soon as it is agreed just what is insignificant. If it is agreed that a variation of 0.1^0 F. is negligible, then at Lexington this will occur at a depth of 46.3 ft. This depth at Ottawa is 31.5 ft.

Figs. 4 and 5 show that when the solar energy is a maximum, both the air temperature and the surface temperature are maximum. Conversely, when the solar energy is minimum, both the air temperature and the surface temperature are minimum. This is to be expected, since the fact that the temperature does not vary at great depths indicates that the source of heat for any variation comes from solar radiation.

It is difficult to observe the effect precipitation has on the soil temperatures. Spring rains generally will be warmer than the ground, and the fall rains will tend to lower the soil temperatures. The normal surface dryness during mid-summer offers resistance to the flow of heat from the surface into the ground, which causes the surface temperature to be exceptionally high during this period of maximum solar radiation. During mid-winter, the moisture that penetrates the soil is limited to a minimum temperature of 32^0 F., and often this penetration is delayed due to the fact that the precipitation is snow,

which must be melted during warmer periods. In localities where snow remains on the surface for long periods of time, it acts as an insulating material between the soil and cold air. All of these mid-winter factors add together to resist and delay the loss of heat from the ground to the air, which does not allow the soil temperatures to drop as much as they would without these factors. These effects are particularly noticeable in the observed surface temperatures for Lexington plotted in Fig. 4.

This method needs verification by use of data from other localities. The equations presented would be more dependable for future calculations if their constants were determined from data covering several years. This is being done for Lexington, since there is now data available covering a period of five years.

Appendix

Sample Calculations

Enough of the calculations for Lexington, Kentucky are presented here that the reader can follow each step that has been outlined for this method.

Calculation of t average.—From the data in Table 1, the value of t average for each depth was calculated by Eq. (2). This equation requires that all of the monthly average values be added together and the sum be divided by twelve. This simple calculation is not shown, but the value of t average for each depth is shown in the bottom line of Table 1.

Calculation of t_m.—Excluding t average at 0 ft. and 2 ft.:

$$t_m = \frac{58.50 + 58.15 + 57.99 + 57.95}{4} = 58.15^{\circ}\text{ F.}$$

Calculation of t_a.—

Depth feet.	t_{x_a} by Eg. 5	$\frac{t_{x_a}}{t_{x_b}}$	$\log_e \frac{t_{x_a}}{t_{x_b}}$	r by Eg. 7
2	$\frac{76.86 - 43.29}{2} = 16.78$	$\frac{16.78}{12.44} = 1.349$	0.2994	0.1497
4	$\frac{71.58 - 46.70}{2} = 12.44$	$\frac{12.44}{10.38} = 1.198$	0.1812	0.0906
6	$\frac{68.96 - 48.19}{2} = 10.38$	$\frac{10.38}{8.59} = 1.208$	0.1889	0.0944
8	$\frac{67.04 - 49.86}{2} = 8.59$	$\frac{8.59}{6.90} = 1.245$	0.2192	0.1096
10	$\frac{65.11 - 51.30}{2} = 6.90$		Σr =	0.4443

by Eq. (8):

$$r_m = \frac{\Sigma r}{N} = \frac{0.4443}{4} = 0.1111$$

$$\log_e \frac{t_{x_a}}{t_{x_b}} = \Delta x \cdot r_m = 2 \cdot 0.1111 = 0.2222$$

$$\frac{t_{x_a}}{t_{x_b}} = 1.249$$

Assume that the value of $t_{10} = 6.90$ is correct as observed.

$$t_8 = 1.249 \cdot t_{10} = 1.249 \cdot 6.90 = 8.62^\circ \text{ F.}$$

$$t_6 = 1.249 \cdot 8.62 = 10.77^\circ \text{ F.}$$

$$t_4 = 1.249 \cdot 10.77 = 13.45^\circ \text{ F.}$$

$$t_2 = 1.249 \cdot 13.45 = 16.79^\circ \text{ F.}$$

$$t_a = t_o = 1.249 \cdot 16.79 = 20.97^\circ \text{ F.}$$

Calculation of α_1 from Amplitude —Solving Eq. (9) for α_1, which is now α_1 in Eq. (13):

$$\alpha_1 = \frac{\pi}{P\,(r_m)^2} = \frac{\pi}{8766\,(0.1111)^2} = 0.0291 \text{ ft}^2/\text{Hr.}$$

Determination of $\Delta\tau$ in Months by Observation.—(See Fig. 6).

a. Plot observed temperatures vs middle of month on a separate sheet for each depth except the surface.
b. Draw a line representing t_m for the entire year on each.
c. Draw a straight line between both sets of successive points that are on opposite sides of the t_m line on each sheet to locate intersections at a and b.
d. Locate the point on t_m that lies half-way between these two straight lines on each sheet.
e. Read the time of the half-way point as accurately as possible on each sheet. This time defines the maximum for each curve.
f. Subtract 3.00 months from each maximum to find when each curve should cross the t_m line.

The above construction was done for the Lexington data from which the following values were determined:

Depth feet	Calc. Crossing of t_m from April 15
2	0.57 mo.
4	1.14
6	1.56
8	2.01
10	2.45

Between Depths of, in feet	Time interval months		$\frac{\Delta\tau}{\Delta X}$ in mo/ft
2 and 10	1.88		0.235
4 and 10	1.31		0.218
6 and 10	0.89		0.222
8 and 10	0.44		0.220
		Total	0.895
	$\frac{\Delta\tau}{\Delta X}$	Average	0.224

Calculation of τ_0.—Total time-lag between surface and 10 ft. is:

$$\text{Total Lag} = \frac{\Delta\tau}{\Delta X} \cdot 10 \text{ ft.} = 0.244 \cdot 10 = 2.24 \text{ mo.}$$

At a depth of 10 ft., the curve crosses the mean at a calculated date of April 15 + 2.45 months. Assuming this value correct, the surface temperature crosses the mean at:

$$(\text{Apr. } 15 + 2.45) = 2.24 = \text{Apr. } 15 + 0.21 \text{ months}$$

or

$$\tau_0 = 0.21 \text{ months} = 6.30^\circ$$

Calculation of α_2 from Time-Lag.—Substituting in Eq. (12)

$$\alpha_2 = \frac{36}{8766(0.224)2} = 0.0261 \text{ ft}^2/\text{hr.}$$

Solving Eq. (13):

$$\sqrt{\frac{\pi}{\alpha_1 P}} = \sqrt{\frac{\pi}{0.02905 \cdot 8766}} = 0.111094$$

Depth	$-x\sqrt{\frac{\pi}{\alpha_1 P}}$	$t_a\, e^{-x\sqrt{\frac{\pi}{\alpha_1 P}}}$
0	0	20.97
2	.22219	16.79
4	.44438	13.45
6	.66656	10.77
8	.88875	8.62
10	1.11094	6.90

$$\sqrt{\frac{\pi}{\alpha_2 P}} = \sqrt{\frac{\pi}{0.02614 \cdot 8766}} = 0.11709$$

Depth	$-x\sqrt{\frac{\pi}{2P}}$ in radians	$-x\sqrt{\frac{\pi}{2P}}$ in degrees
0	0	0
2	-0.23418	13.4
4	-0.46836	26.8
6	-0.70254	40.2
8	-0.93672	53.7
10	-1.17090	67.1

Only the calculations for a depth of 2 feet are shown below, since this is typical of all depths.

For x = 2 ft., Eq. (14) becomes:

$$t_{x=2} = 58.15 + 16.79 \sin (30\tau - 6.3 - 13.4)$$

Date	$30\tau - 19.7$	Sin $(30\tau - 19.7)$	Times 16.79	+ 58.15
Apr. 15	-19.7	-0.3371	- 5.66	52.49
May 15	10.3	0.1788	3.00	61.15
June 15	40.3	0.6468	10.86	69.01
July 15	70.3	0.9415	15.81	73.96
Aug. 15	100.3	0.9839	16.52	74.67
Sept 15	130.3	0.7627	12.81	70.96
Oct. 15	160.3	0.3371	5.66	63.81
Nov. 15	190.3	-0.1788	- 3.00	55.15
Dec. 15	220.3	-0.6468	-10.86	47.29
Jan. 15	250.3	-0.9415	-15.81	42.34
Feb. 15	280.3	-0.9839	-16.52	41.63
Mar. 15	310.3	-0.7627	-12.81	45.34

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Table 1. Observed Data for Lexington, Kentucky for 1953

Month	Average Soil Temperature in Degrees, F. Depth Beneath Surface (ft)						Average Solar Energy B/ft[2] Day	Mean Air[a] Temp. in Degrees, F	Precipitation[a]		
	0	2	4	6	8	10			Rain In.	Snow In.	Total In.
Jan.	41.41	44.56	47.62	49.92	52.44	54.87	561.68	38.5	4.12	1.50	5.62
Feb.	40.16	43.29	46.70	48.85	51.04	53.24	928.57	38.9	0.92	1.8	2.72
Mar.	49.79	47.39	47.30	48.19	49.86	51.60	1415.63	44.6	7.26	0.1	7.36
Apr.	56.49	51.73	50.14	49.91	50.39	51.30	1668.76	54.7	2.06	T	2.06
May	68.95	60.74	56.07	53.88	52.84	52.70	2117.62	65.2	3.15	T	3.15
June	85.36	72.79	64.92	60.93	57.97	56.21	2454.58	78.8	4.43	T	4.43
July	84.51	76.85	70.16	65.87	62.26	59.69	2466.06	79.7	3.01	0.0	3.01
Aug.	79.97	75.48	71.58	68.50	65.43	62.87	2097.79	76.6	2.80	0.0	2.80
Sept	72.69	71.39	70.54	68.96	67.04	64.88	1950.87	68.4	1.58	0.0	1.58
Oct.	55.42	61.87	65.82	66.37	66.16	65.11	1388.16	51.1	1.17	T	1.17
Nov.	46.04	53.27	58.96	61.05	62.46	63.18	843.61	45.9	1.41	2.1	3.51
Dec.	39.04	45.70	52.22	55.38	58.07	59.75	521.65	38.3	1.48	2.1	3.58
Av.	59.90	58.76	58.50	58.15	57.99	57.95	1534.60	56.68			

a. Recorded by U. S. Weather Bureau at Blue Grass Airport, Lexington, Kentucky.

T. Trace, an amount too small to measure.

Table 2. Calculated and Observed Soil Temperature at Lexington, Kentucky during 1952.

Month	Temperature in Degrees, F at depths of											
	0 ft.		2 ft.		4 ft.		6 ft.		8 ft.		10 ft.	
	Calc.	Obs.	Calc.	Obs.	Calc.	Obs.	Calc.	Obs.	Calc.	Obs.	Calc	Obs.
Jan.	37.31	41.41	42.34	44.56	46.88	47.62	50.74	49.92	53.84	52.44	56.18	54.87
Feb.	38.95	40.16	41.63	43.29	44.72	46.70	47.82	48.85	50.69	51.04	53.14	53.24
Mar.	45.74	49.79	45.34	47.39	46.16	47.30	47.68	48.19	49.53	49.86	51.14	51.60
Apr.	55.85	56.49	52.49	51.73	50.80	50.14	50.34	49.91	50.69	50.39	51.54	51.30
May	66.58	68.95	61.15	60.74	57.42	56.07	55.09	53.88	53.84	52.84	53.41	52.70
June	75.03	84.36	69.01	72.79	64.23	64.92	60.66	60.93	58.15	57.97	56.55	56.21
July	78.99	84.51	73.96	76.86	69.42	70.16	65.56	65.87	62.46	62.26	60.12	59.69
Aug.	77.35	79.97	74.67	75.48	71.58	71.58	68.48	68.50	65.61	65.43	63.16	62.87
Sept	70.56	72.69	70.96	71.39	70.14	70.54	68.62	68.96	66.77	67.04	64.86	64.88
Oct.	60.45	55.42	63.61	61.87	65.50	65.82	65.96	66.37	65.61	66.16	64.76	65.11
Nov.	49.72	46.04	55.15	53.27	58.88	58.96	61.21	61.07	62.46	62.46	62.89	63.18
Dec.	41.27	39.04	47.29	45.70	52.07	52.22	55.64	55.38	58.15	58.07	59.75	59.75

Table 3. Observed Data for Ottawa, Ontario for 1950

Month	Average Soil Temperature [a] in Degrees F, Depth Beneath Surface (ft)					Average [b] Solar Energy B/ft² Day	Mean [b] Air Temp. °F.	Precipitation [b] Total in.
	0	1	2	4	8			
Jan.		32.6	34.9	40.2	46.0	379	19.8	3.12
Feb.		30.6	33.6	38.3	44.5	858	11.6	2.87
Mar.		30.8	33.0	36.7	43.1	1427	19.8	2.50
Apr.		35.0	34.6	36.6	41.8	1460	37.4	2.41
May		49.7	45.8	41.9	41.6	1755	54.1	2.03
June		58.2	57.2	50.1	43.9	1971	64.0	2.73
July		66.0	64.1	56.3	47.5	2013	68.2	3.93
Aug.		63.6	63.4	58.8	50.5	1550	64.1	4.33
Sept		56.9	58.8	57.9	52.3	1347	54.2	1.34
Oct.		48.2	51.6	54.1	52.7	752	47.6	1.94
Nov.		40.2	43.5	48.4	51.4	344	35.4	4.39
Dec.		34.8	38.1	42.8	48.4	464	18.2	2.80
Av.		45.55	46.55	46.84	46.98	1193	41.53	2.87

a. From reference 6. Used with permission.

b. Supplied by Department of Transport, Meteorological Division, Totonto, Ontario.

Table 4. Calculated and Observed Soil Temperatures of Ottawa, Ontario for 1950

Month	Temperature in Degrees F at Depths of									
	0 ft. Calc.	0 ft. Obs.	1 ft. Calc.	1 ft. Obs.	2 ft. Calc.	2 ft. Obs.	4 ft. Calc.	4 ft. Obs.	8 ft. Calc.	8 ft. Obs.
Jan.	25.28		29.20	32.6	32.83	34.9	38.93	40.2	46.12	46.0
Feb.	27.21		29.25	30.6	31.54	33.6	36.23	38.3	43.50	44.5
Mar.	34.42		34.03	30.8	34.36	33.0	36.39	36.7	41.76	42.1
Apr.	44.98		42.26	35.0	40.55	34.6	39.37	36.6	41.42	41.8
May.	56.05		51.74	49.7	48.44	45.8	44.37	41.9	42.55	41.6
June	64.68		59.92	58.2	55.93	57.2	50.05	50.1	44.85	43.9
July	68.54		64.62	66.0	60.99	64.1	54.89	56.3	47.70	47.5
Aug.	66.61		64.57	63.6	62.28	63.4	57.59	58.8	50.32	50.5
Sept.	59.40		59.79	56.9	59.46	58.8	57.43	57.9	52.06	52.3
Oct.	48.84		51.56	48.2	53.27	51.6	54.45	54.1	52.40	52.7
Nov.	37.77		42.08	40.2	45.38	43.5	49.45	48.4	51.27	51.4
Dec.	29.14		33.90	34.8	37.89	38.1	43.77	42.8	48.97	48.4

FIG. I. PHOTOGRAPH OF PYRHELIOMETER MOUNTED ON TOP OF ANDERSON HALL AT THE UNIVERSITY OF KENTUCKY. LATITUDE 38° 42' N; LONGITUDE 84° 30' W; ELEVATION 1026.6 FT.

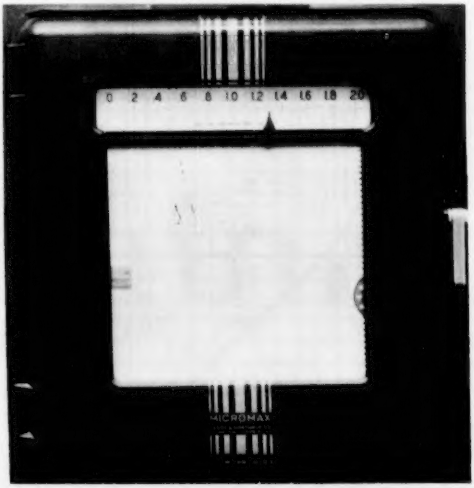

FIG. 2. STRIP CHART RECORDER USED TO RECORD SOLAR RADIATION DATA.

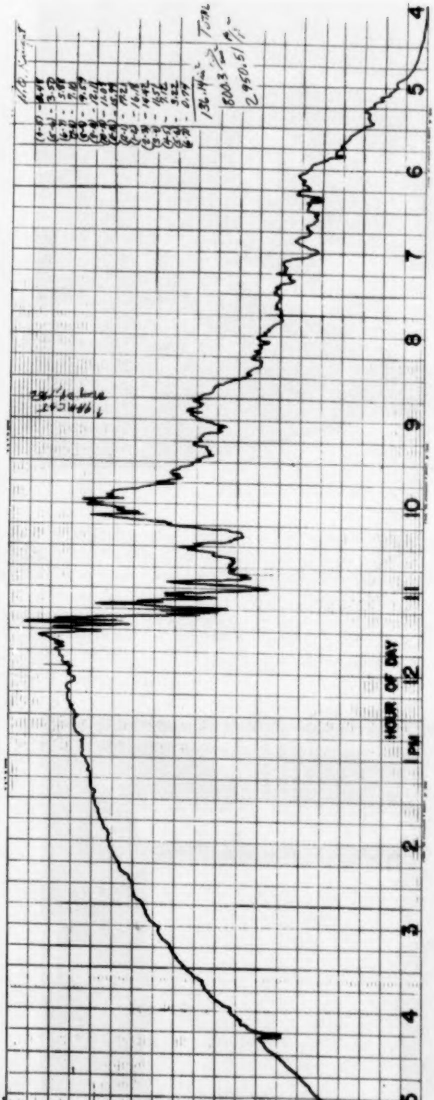

FIG. 3. STRIP CHART SHOWING CURVE OF SOLAR ENERGY FOR A TYPICAL DAY.

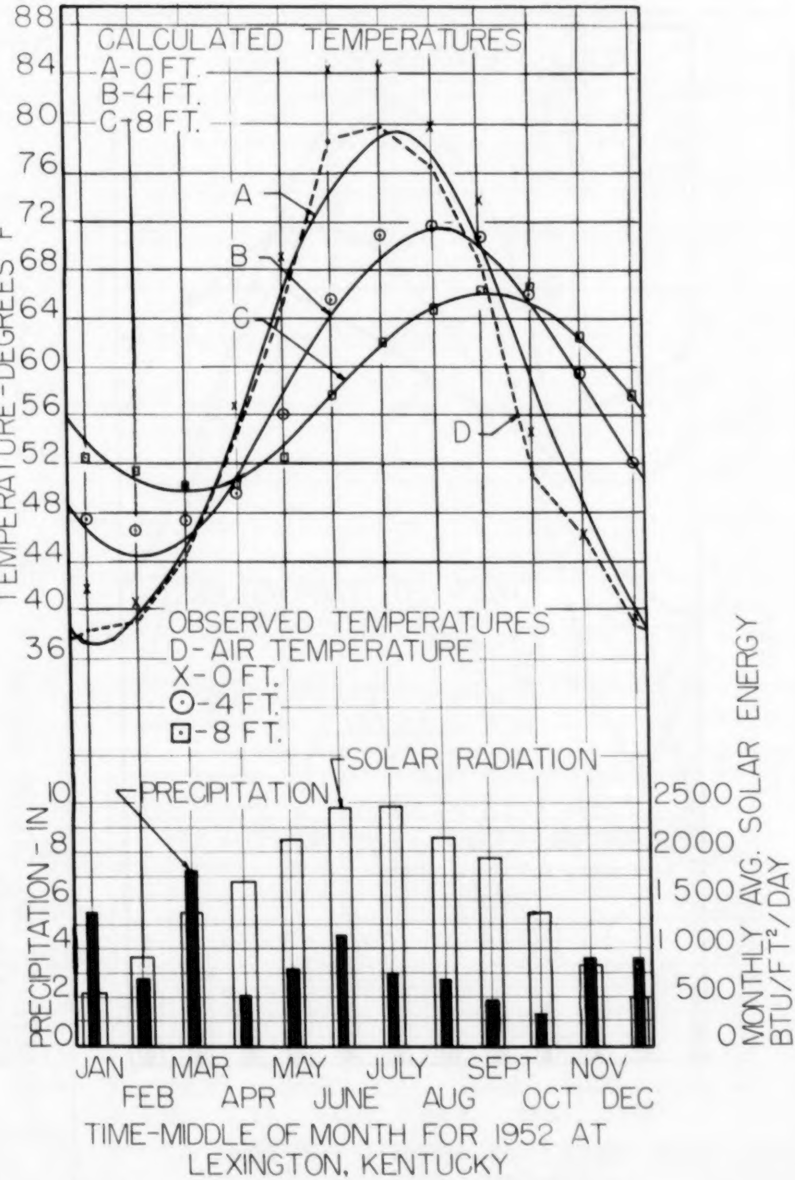

FIG. 4. COMPARISON OF CALCULATED AND OBSERVED TEMPERATURES AT LEX., KY.

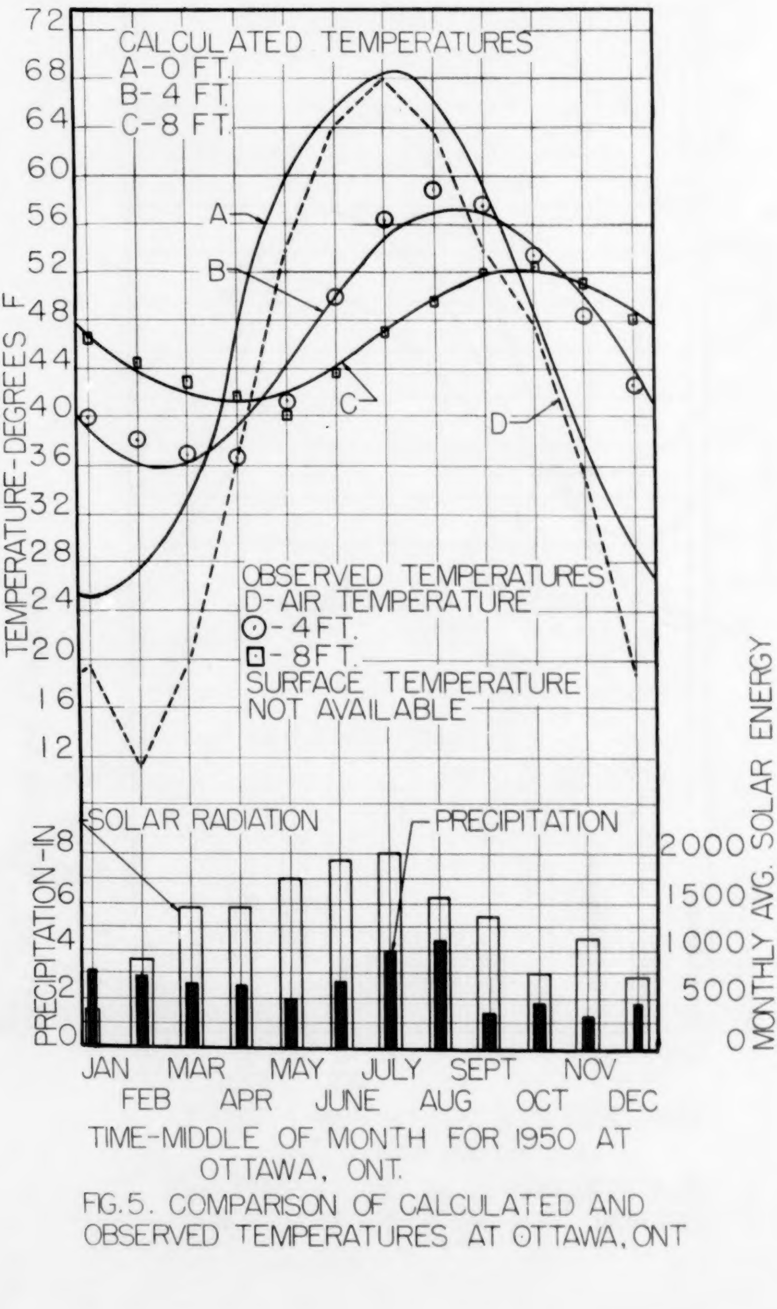

FIG. 5. COMPARISON OF CALCULATED AND OBSERVED TEMPERATURES AT OTTAWA, ONT

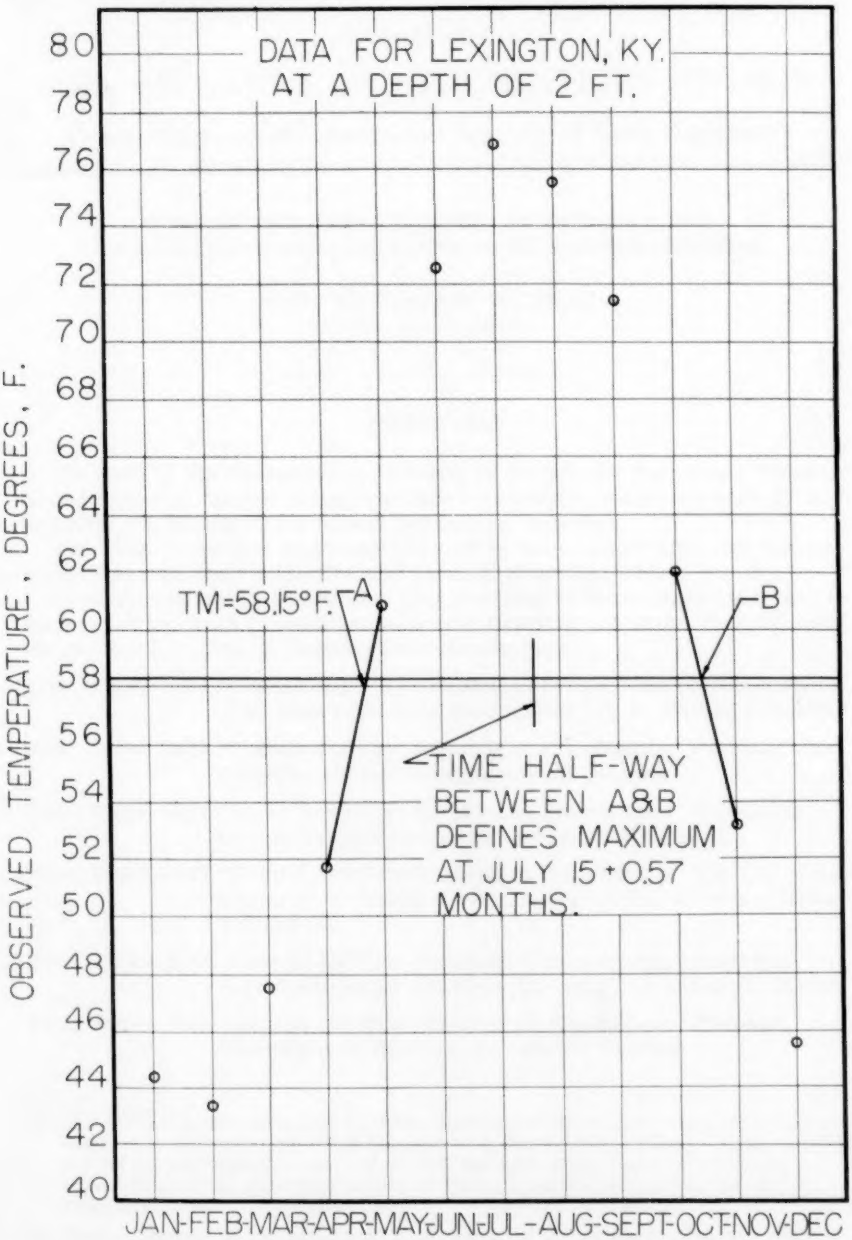

FIG. 6. GRAPHICAL DETERMINATION OF TIME THAT CURVE REACHES A MAXIMUM.

Paper 1544 February, 1958 SM 1

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

CEMENT AND CLAY GROUTING OF FOUNDATIONS: PRESENT STATUS OF PRESSURE GROUTING FOUNDATIONS

A. Warren Simonds,[1] M. ASCE
(Proc. Paper 1544)

FOREWORD[a]

The work of the Committee on Grouting of the Soil Mechanics and Foundations Division is divided among four task committees, concerned with (1) bituminous, (2) chemical, (3) cement and (4) clay grouting.

The Task Committee on Chemical Grouting has completed its task and its report was published in the Division Journal, November 1957.

"The Symposium on Cement and Clay Grouting of Foundations" is a "joint venture" of the Task Committee on Cement Grouting and of the Task Committee on Clay Grouting. It includes the following papers:

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Note: Discussion open until July 1, 1958. A postponement of this closing date can be obtained by writing to the ASCE Manager of Technical Publications. Paper 1544 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SM 1, February, 1958.

1. Engr., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.
a. By Raymond E. Davis, Chairman, Committee on Grouting.

Proc. Paper 1550 "Cement and Clay Grouting of Foundations: French Grouting Practice" by Armand Mayer

Proc. Paper 1551 "Cement and Clay Grouting of Foundations: Practice of the Corps of Engineers" by Edward P. Burwell, Jr.

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These papers are based largely on the results of successful grouting operations for a wide variety of foundation conditions but also are based on the results of laboratory investigations and to some extent on the ideas and opinions of designing engineers who are concerned with foundation problems.

The information contained in the papers has been assembled for the particular benefit of those responsible for the foundation treatment of structures. Recognizing that these papers by no means represent the "last word" in good practice and that there may be many others concerned with foundations who could contribute to our knowledge of cement and clay grouting, discussion is earnestly sought.

ABSTRACT

Present day uses of pressure grouting are described. Recent developments and improvements of drilling and grouting equipment and also of grouting materials are mentioned. Examples of successful grouting are cited where core drilling has produced rock cores from foundations with cracks and seams well filled and bonded with cement grout.

INTRODUCTION

Pressure grouting is the process of injecting suitable cementitious slurries or similar materials into inaccessible places, such as the underlying formations of foundations of structures for the purpose of sealing seams, cracks, and fissures, or filling voids. While the principal use of this process is to fill openings in a structural mass and render it impervious to percolating water, it is also used to improve the strength and elastic properties of the material into which it is injected. This process has been used extensively in the construction of a variety of structures such as dams, buildings, and tunnels, and in a wide range of special cases.

In the construction of concrete dams, the principal purpose of foundation grouting is to establish an effective underground barrier against flow of water, thereby preventing leakage and reducing the hydrostatic uplift pressure under the structure. A second purpose is to fill voids in the near surface rock under the structure and thus secure a more uniform and monolithic foundation. Prominent examples of the efficacy of pressure grouting during recent years can be cited.

At Hoover Dam an extensive program of grouting was used to control seepage and leaks, and to reduce the hydrostatic uplift pressure on the base of the

structure.(1) The effect of the grouting on the uplift pressure gradient is shown diagrammatically in Figure 1. The program of additional grouting which established a deeper cutoff beneath the dam (additional A-hole in figure), took place between September 1938 and August 1949. The magnitudes of the observed uplift pressure before and after the grouting program are indicated on the section of the dam shown in the figure. The decrease in uplift pressure is self-evident.

This grouting program had a pronounced effect on the discharge of the foundation drains. Before the grouting was started, the discharge flow from the drains for the maximum reservoir water surface elevation in 1938 was 2,030 gallons per minute. After the grouting had been completed, the discharge from the drains was 135 gallons per minute for the maximum reservoir water surface elevation in the 1949 season.

At Norris, Guntersville and Fontana Dams of the TVA, pressure grouting was a necessary process in correcting foundation defects where solution channels and voids in the foundation were encountered. The grouting program at Norris Dam required unusually large quantities of materials.(2) A total of 202,770 cubic feet of cement grout was used in treating the foundation of the dam and its appurtenant structures, while 257,736 cubic feet of cement-rock flour grout were used in grouting the reservoir rim. At Guntersville Dam 47,482 cubic feet of cement and 15,793 cubic feet of sand were required for grouting the foundation; and at Fontana Dam, 42,817 cubic feet of cement was required.(3)

Of all the varied types of foundation conditions encountered at large dam sites, one of the most difficult types to treat adequately is that composed of layers of uncemented shale or siltstone and sandstone such as the foundation of Conchas Dam which was built by the Corps of Engineers.(4) Of the many problems involved in the construction of this dam, one of the difficult problems was the control of seepage under the dam through an artesian sandstone, and of the uplift pressure on the foundations of the dam from artesian water in this formation. This problem was solved by a program of grouting and drainage in which a total of 61,647 cubic feet of cement was injected in the foundation.

In the construction of earth dams, pressure grouting has been used to form cutoff curtains beneath the impervious sections of the dams, to fill the cracks in the near surface rock, and to dry up springs and seeps in the foundation area which would impede the placing of the fill materials. At such dams as Island Park, Anderson Ranch, Fort Peck, Clear Water, Savage River, Crooked Creek, Yohiogheny, and many others, pressure grouting has been used advantageously to improve deficiencies in the foundation rock.

For the improvement of foundations for buildings, pressure grouting has been used for a great many years. One of the most interesting examples of treating a deficient foundation in recent years, although extremely controversial, was the grouting done in an attempt to stabilize the foundation of the leaning tower of Pisa in Italy. Precise measurements made on the tower indicated that its lean was increasing at the rate of 0.04 inch per year. In 1932, an attempt was made to stabilize its foundation by injecting over 1,000 tons of cement and sodium silicate grout into the underlying soil through holes drilled into the foundation. This checked the lean until World War II when the foundation was disturbed by bombs dropped in the vicinity.(5)

Grouting has been used to strengthen the foundations of buildings housing heavy equipment subject to vibration. The process was used to improve the

faulted limestone foundation for the building of the Clinton Engineering Works in which the heavy electro-magnetic equipment used in the process of separating uranium isotopes, is now housed. Oil and gas transmission companies have used cement grouting to stabilize and reduce vibration of foundations of their pipeline compressor plants where they have been located on alluvial formations.(6,7)

An unusual case of foundation treatment for a building was the grouting program at the Veterans Administration General Medical Hospital in Pittsburgh, Pennsylvania. This eleven-story building was built on a foundation of sandstone, clay, and shale above an old coal mining area. Holes were drilled at a spacing of 10 feet each way for consolidating the upper surface area of the foundation. Some abandoned coal mining tunnels passed beneath the structure at depths of 100 to 200 feet. Large quantities of proprietary grout, of which cement was one of the principal ingredients, were used in backfilling these tunnels.(8)

In tunnel construction, pressure grouting has been used advantageously at many locations. Grouting ahead of the excavation of the tunnel bore has been done to shut off flows of water into the working area and to solidify caving ground. It has been used to consolidate the rock immediately surrounding the bore. Where tunnels are lined with concrete, grouting has been used extensively to fill the voids at the top of the arch between the concrete lining and the rock. Pressure grouting has been used in the construction of the tunnels of the Catskill aqueduct, the Delaware aqueduct, the San Jacinto tunnel, the Tecolote tunnel, the power tunnels of Ontario Hydro Electric Power Commission.

In addition to these examples, pressure grouting has been used for many special purposes. Abutments of dams have been staunched by mechanical doweling or tieing, and the intervening joints and seams grouted.(9) An outstanding example of this was the treatment of the abutments at Castillon Dam in France as shown in Figure 2.

Difficulties arising from equipment failures which formerly plagued engineers and contractors have been reduced materially in recent years as the result of improvements in grout pumps and mixers. It was only a few years ago when the customary practice was to provide several standby grout pumps for use at each hole being grouted. At present, as the result of improved design of grout pumps and better accessories, the practice of providing standby equipment is becoming less prevalent.

The duplex piston-type pump with special fittings for cement grout service has been the most popular type of pump for pressure grouting. While this type of pump is satisfactory for pressure grouting, there are other types of positive displacement pumps which may prove to be more suitable. One type of pump which may even supplant the positive displacement pumps is the centrifugal pump specially equipped for grout service.

One necessary operation in connection with pressure grouting is the drilling of the grout holes. There has been a divergence of opinion among engineers as to the relative value of percussion drilled holes and rotary drilled holes for grouting. This difference of opinion is due to the tendency for cuttings from the percussion drills to plug small seams and cracks. For this reason some specifications prohibited the use of percussion drills for grout hole drilling. Recently steel bits for rotary drills which enable the drilling of dry holes have been developed and may lower the cost of drilling considerably in certain formations.

A recent development by the Corps of Engineers is the Borehole Camera.(10) This camera is small enough to be lowered into a NX-size hole having a diameter of 3 inches. By means of this camera a continuous undistorted cylindrical color picture of dry or water-filled borings can be obtained. For the geological investigation of bedrock imperfections where pressure grouting is to be used, this camera offers a great saving in costs and time involved as compared with the old style method of sinking large diameter calyx holes.

New materials for pressure grouting are developed from time to time. These consist chiefly of additives and admixtures to be used with cement or clay grouts. They can be obtained for special purposes such as controlling the time of set, improving the pumpability of the grout, or to serve as a bulking component for the grout.

In the process of pressure grouting, when grout is being injected into an underlying formation where it cannot be seen, there is always some speculation as to where the grout is going and how effective this procedure is. Oftentimes when unusually large quantities of grout have been injected through a single drill hole, a core hole will be drilled close by to check the efficacy of the grouting. In many cases no grout can be found in the core recovered. In such cases this exploratory drilling was done too soon after grouting to obtain a core with a grout film. Unless the injected grout has had time to develop some strength, any grout encountered in the drill hole may be lost due to grinding in the drilling process and washed away. However, in many cases excellent seams have been found by core drilling.

Typical examples of the results of pressure grouting of rock foundations are shown in Figures 3 to 5, inclusive. Figure 3 shows a core 2-1/8 inches in diameter obtained from drilling one of the foundation drains at Grand Coulee Dam. The two edges of the grout seam in the core extend lengthwise in the specimen of the granite foundation rock. This core was obtained at a depth of about 48 feet into the foundation. The grouting in this area was done with pressures between 200 and 400 psi, and the water cement ratios of the grout varied from 4:1 to 2:1. Figure 4 shows a seam almost 1 inch wide filled with grout which was uncovered during the excavation of the foundation for the pumping plant at the left abutment of Grand Coulee Dam. The 1-inch section of the folding rule lays above the grout seam in the figure. Figure 5 shows a core containing a film of grout drilled from the andesite foundation rock at Hoover Dam.

From these and other examples of grouted seams and cracks which have been obtained from many foundations where cement grouting was used, the conclusion may be made that pressure grouting can be used advantageously as a means of improving rock foundations. In many cases, however, realization should be made that there are other materials which are equally as satisfactory as cement for grouting under certain conditions. It is only fitting, therefore, that this introduction be followed by a discussion of the use of clay in grouting operations.

The development of suitable materials for pressure grouting is a continuing problem. While the basic materials for grouting are cement, clays, and chemicals, new ingredients and new combinations appear from time to time and a discussion of admixtures in cement grouts is included in the symposium.

Due to certain intangible aspects of pressure grouting, specifications for this work have been somewhat of a controversial nature. In order to obtain effective treatment of the foundation by grouting and to be fair to both the

owner and contractor, grouting specifications are difficult to prepare without developing controversial provisions. As a part of this symposium, suitable specifications for pressure grouting are discussed with emphasis on eliminating ambiguities.

Information on the most recent developments and practices is presented in the following papers. The packer grouting method presently used by the Bureau of Reclamation, the latest French practices, the present practice in cement grouting as developed by the Corps of Engineers, and the experience of the TVA with clay cement and related grouts complete the symposium.

In summarizing the contents of this symposium, the most recent available information on pressure grouting procedures, techniques and materials is presented. In time new developments will take place, new materials will be available and many foundations which are now considered questionable will be improved so that they can be utilized for construction projects.

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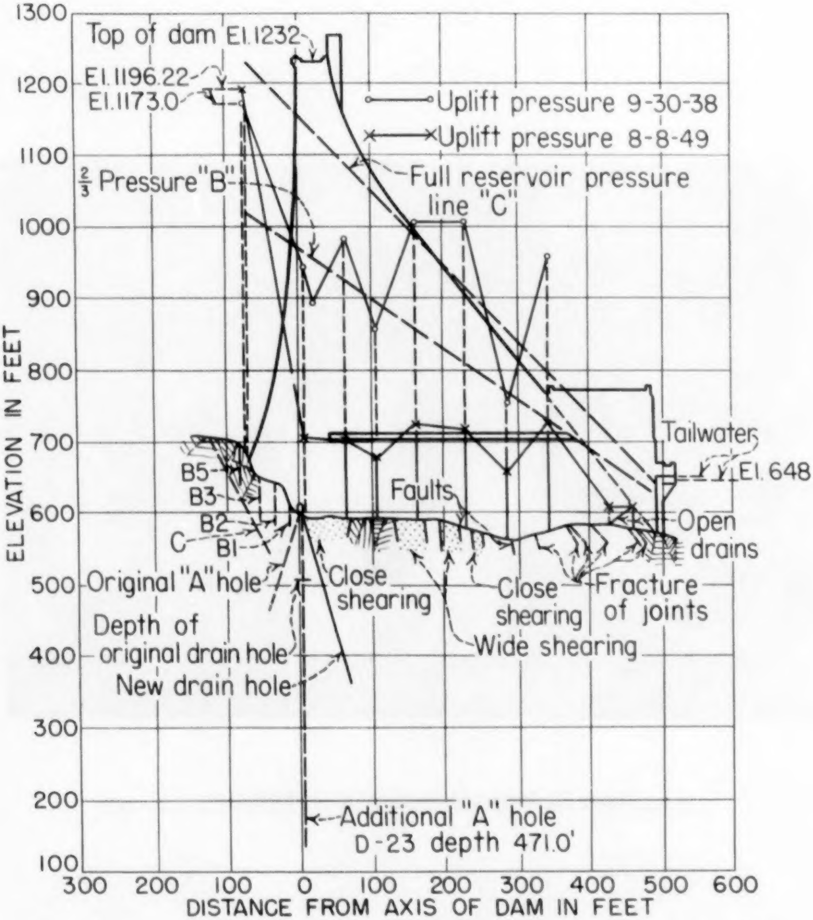

FIGURE 1-UPLIFT PRESSURE GRADIANT-SET B

SECTION ALONG 10-FT. GALLERY
FROM ELEVATOR TO POWER HOUSE
HOOVER DAM - NEVADA SIDE

Figure 2. Staunched Abutment at Castillon Dam

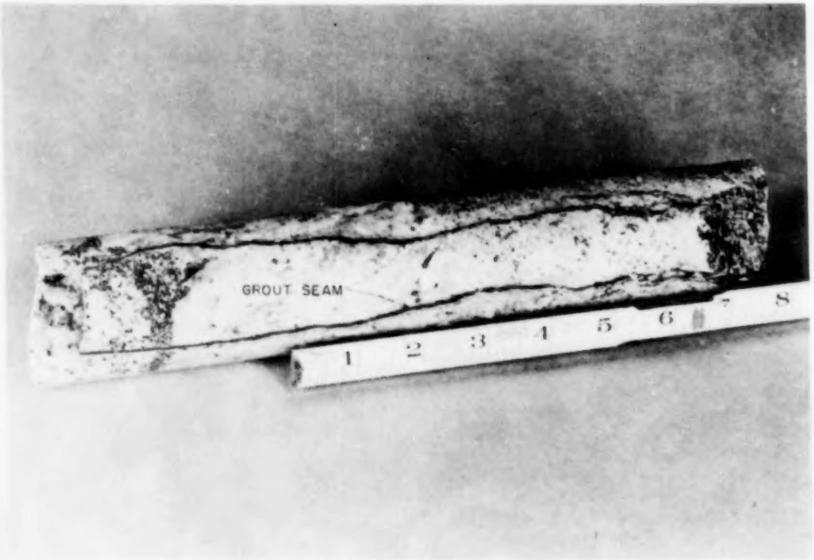

Figure 3. Granite Core from Grand Coulee Dam Showing Grout Film

Figure 4. Excavated Abutment at Grand Coulee Dam Showing Grout Seam

Figure 5. Grout Seam 3/4" Thick in Core from Hoover Dam Foundation

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CEMENT AND CLAY GROUTING OF FOUNDATIONS: GROUTING WITH CLAY-CEMENT GROUTS

Stanley J. Johnson,[1] M. ASCE
(Proc. Paper 1545)

FOREWORD[a]

The work of the Committee on Grouting of the Soil Mechanics and Foundations Division is divided among four task committees, concerned with (1) bituminous, (2) chemical, (3) cement and (4) clay grouting.

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1. Associate, Moran, Proctor, Mueser & Rutledge, New York, N. Y.

a. By Raymond E. Davis, Chairman, Committee on Grouting.

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ABSTRACT

The uses of suspension grouts consisting of soil or soil and cement are reviewed together with a discussion of the design of grout mixtures.

INTRODUCTION

Pressure grouting using materials other than neat cement has been practiced for a long time but generally has been regarded as a substitute for neat cement grouting. In recent years, however, other types of grouts have been regarded more on the merits of their suitability to the proposed work and less as merely a substitute for neat cement grout. The use of soils as grouting materials in soil-water mixes, or with the addition of cement, is similar in many respects to the use of neat cement grout since these are all basically suspension grouts in which solid material is suspended in water.

Uses

The principal uses of suspension grouts consisting of soil-water or soil-cement water suspensions fall into three main categories, as follows: (1) filling voids which are so large in comparison with the suspended grouting materials that penetration of the grout occurs freely; (2) filling small voids in which penetration of the grouting materials is just barely possible, or may not be possible if the solid particles are large; and (3) compaction grouting in which the grout under pressure fills relatively large voids and also compresses adjacent soft materials. The conditions to be grouted often vary greatly even on the same job and must be studied carefully in order to select grouting materials which are suitable. More than one type of grout may be

necessary for maximum economy and efficiency of results.

Joints and fractures in rock may be so large that grouting is primarily a backfilling operation which can be done with a choice of many materials or they may be so small that the size of the suspended solid particles in the grout becomes of primary importance. The above cases also may be found in grouting solution voids and channels, in grouting coarse sands and gravels and in grouting to compensate for loss of ground and for other purposes. Grouting of soil foundations containing voids and soft spots or zones caused by saturation of a low density material such as loess involves filling large voids and compaction of the adjacent natural softened material.

The preceding general conditions requiring grouting are encountered in foundations for dams, buildings, powerhouses, tunnels, cofferdams, railroad ballast pockets and structures of many types. When the primary purpose of grouting is to reduce the flow of water through soil or rock the permeability and permanence of the grout and its ability to penetrate the formation are important. When grouting is undertaken to increase the structural strength of the foundation the strength of the grout becomes important. There are, however, many cases where grouting of foundations beneath structures is undertaken to fill voids in the overburden or in rock caused by defects due to solution or other causes and in which the strength requirements of the grout are so nominal that the necessary strength is easily obtained with one of a number of different types of grout.

Grouting Materials

Many naturally occurring materials have been successfully used in grouts such as clays and silts suspended in water; and clays, silts and sand suspended in water with the addition of cement and possibly other additives. The use of suspension grouts requires a thorough knowledge of the formation to be grouted and of the properties of grouts which may be applicable. Laboratory and field tests are very helpful in designing grout mixes to suit the conditions being grouted and the materials available and are recommended unless the job is small.

The grain-size distribution curves of typical soils commonly used in soil-cement grouts is shown on Figure No. 1. It can be seen that these materials range from fine sands to silty clays. Typical soils used in soil-water grouts are shown on Figure No. 2 and are finer grained than those used in soil-cement grouts. Soil-cement grouts are normally sufficiently impervious for almost any purpose because of the cementing action and are strong enough to be used where a strength equivalent to a strong soil or a weak rock is indicated. Materials for soil-water grouts must be fine-grained in order to be impervious and yet sufficiently pervious so that excess water can be driven off. The soil used in the extensive silt injection grouting performed by the Tri-County Project in Nebraska (Central Nebraska Public Power and Irrigation District) under the direction of George Johnson is shown on Figure No. 3.

Penetration

The ability of a grout to penetrate the voids of a formation is of paramount importance in considering grouting of pervious foundations to decrease seepage beneath structures or into cofferdamed areas. This type of grouting is

receiving more attention and the possibilities for successful grouting for this purpose are constantly increasing. While the penetration of suspension grouts depends on the nature of the contact surface of the solid particles and their concentration and other similar features, the primary factor involved is the size of the voids being grouted as compared to the size of the solid particles in the grout. Tests show that it is the smallest voids in the formation being grouted which stop penetration of the grout and likewise that the largest sizes of the solid particles in the grout prevent the finer sizes from entering the formation being grouted. These factors suggested that a "groutability-ratio" might be developed relating the properties of the formation to those of the solid particles in the grout. This concept is actually a simple reverse extension of the filter criteria originally developed by Terzaghi, although separate definitions of the "groutability-ratio" are needed for grouting openings of a fixed size, such as fissures, and voids in a natural formation such as coarse sand and clean gravel. The voids in a sand or gravel are related in a general way to the grain-sizes of the individual particles so that a "groutability-ratio" for grouting natural soil formations may be defined as the ratio of the 15% size of the formation being grouted to the 85% size of the grout. A fairly substantial number of tests indicate that this ratio should be greater than about 25 if the grout is to successfully penetrate the formation being grouted. This figure is approximate and should not be accepted without verification by tests in any particular case but nevertheless it is useful, for illustrating the mechanical concept of the size relationship between the voids of the formation being grouted and the size of the particles in the grout. This requirement illustrates why suspension type grouts are normally limited to grouting medium sands and coarser grained soils. Some fine and fine to medium sands can be grouted with very fine grained materials such as bentonite but these cases should be checked carefully by field and laboratory tests.

Mix Design

Factors which must be considered in designing suspension grouts involve such items as the ability to mix the materials, the proportions to be used, strength required, segregation, pumpability and of course the materials available. If a very plastic clay is used, the mixability may be poor whereas lean clays and clayey sands and silts can be mixed much more easily. Admixtures are used to facilitate penetration, control rate of set and to prevent segregation. For example, in sand-cement grouts it has been found that the addition of a moderate amount of bentonite will greatly reduce the tendency towards segregation and still result in strengths which are satisfactory for many purposes. The addition of fly-ash or silt will also help to reduce segregation. The grout design should emphasize prevention of segregation for any segregation after a grout has been injected results in incomplete filling. This is generally regarded as the main reason why re-grouting often results in a surprisingly large grout acceptance. Chemicals have been used as admixtures to prevent segregation and include sodium silicate, calcium chloride, sodium tripolyphosphate, sodium hexametaphosphate and others.

Typical proportions which have been used vary greatly according to the proposed use of the grout. The best method of expressing grout proportions is on a weight basis but one that is commonly used because of its convenience is a loose volume basis. Using the latter, the loose volume of soil generally

varies between one and 12 times the loose volume of cement. Volumes of soil between about four and six times the loose volume of cement are the most common. These proportions have been used for sands, solts and clays.

The volume of water which is added depends greatly upon the thinking of the engineer in charge and varies between wide limits. For example, as little as three cubic feet or as much as ten cubic feet of water has been added to a one to four (by loose volume) cement-clay grout. The volume of water which has been used in soil-cement grouts is indicated on Figure No. 4 for clay-cement and for sand-cement grouts. Bentonite was added to reduce segregation of the sand-cement grouts shown. It can be seen that the volume of mixing water for clay-cement grouts varies from about 3/4 to twice the loose volume of clay used per bag of cement and for sand-cement grouts varies from about 1/3 to one times the loose volume of sand used per bag of cement. Clay-water or silt-water grouts are normally proportioned with a volume of mixing water equal to about one to two times the volume of loose soil.

Some engineers prefer a thin mix because they contend that penetration of smaller openings or voids is facilitated and that excess water in the grout is driven off into the voids or formation being grouted. These engineers point out that solid particles in suspension grouts are filtered out by small voids through which water can move and further that practically all suspension grouts, including neat-cement, are relatively pervious during the grouting operations, thereby facilitating the driving off of excess water. The amount of water driven off increases with the grouting pressure and with the length of time the pressure is maintained. Laboratory tests have shown that many grouts considered satisfactory on the basis of field usage lose as much as 35 to 50% of their as-mixed volume when normal grouting pressures are used, and from 10 to 50% when no pressure is applied. These figures are typical of both neat-cement and of clay-cement and sand-cement grouts.

Many other engineers, equally experienced in grouting work, contend that grout should contain only enough water to transport the solids and to insure sufficient fluidity to fill up voids and cracks. These engineers contend that the free water on top of a specimen of grout after setting and without pressure applied should preferably be zero and should not exceed 5% of the as-mixed volume. This requirement can normally be obtained without difficulty. While both viewpoints represent the thoughts of much practical grouting experience, there is a tendency towards the use of grouts with a minimum water content.

The quantity of mixing water in soil-water grouts should be sufficient to result in a thick creamy mixture but no more than this amount should be used.

Strength

The compressive strength of a grout is generally determined on the basis of the grout as mixed in the laboratory or in the field, which neglects the loss of excess water and resultant increase in strength which generally occurs during injection. The unconfined compressive strengths of typical clay-cement and sand-cement grouts are shown on Figure No. 5 from which it can be seen that adequate strengths can be obtained for most purposes and that high strengths can be obtained if needed. Typical compressive strengths of sand-cement grout are tabulated on Table I. Since the strengths shown are unconfined compression strengths it is apparent that the mixes shown are satisfactory for many purposes even with the addition of bentonite.

Grouting Pressures

The grouting pressure should be as high as the job conditions will permit but should not be excessive. Pressures in excess of those which can be taken by the formation will simply open up or create joints, forcing stringers of grout into the formation, and will do more harm than good as well as waste grout by pushing it beyond the area where grout is desired. As a consequence, there has developed a tendency to use lower pressures when using soil-cement grouts. As high pressures as possible are used with silt or clay water mixes to force excess water out of the grout.

Field Grouting Equipment and Techniques

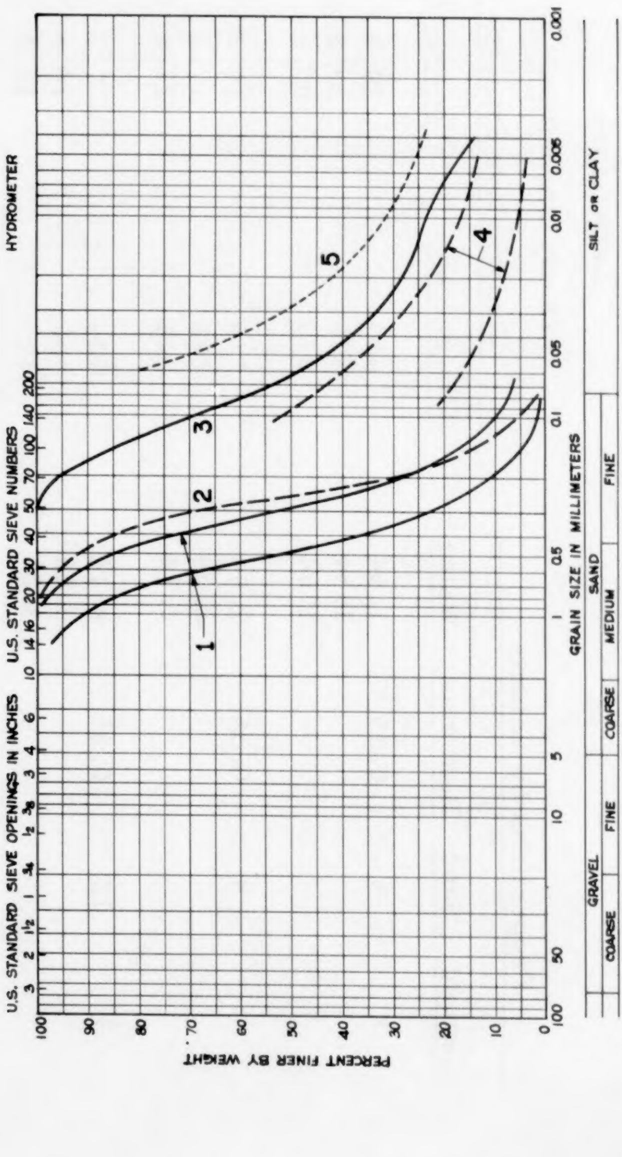
Grouting with soil or soil-cement grouts differs in some respects from grouting with neat cement grout but generally similar pumps are used, though the mixing facilities are larger. Sometimes larger capacity pumps are used, as in silt-injection grouting. Soil-cement grouts do not set up rapidly as do neat cement grouts and grouting can be continuous or it can be intermittent without the danger of losing the hole as with neat cement grouting. It is often not possible to use packers and various alternates must be used, such as sand and gravel seals around the bottom of injection pipes and similar measures.

TABLE NO. 1

UNCONFINED COMPRESSIVE STRENGTHS - TYPICAL SAND-CEMENT GROUT

(After C. S. Proctor)

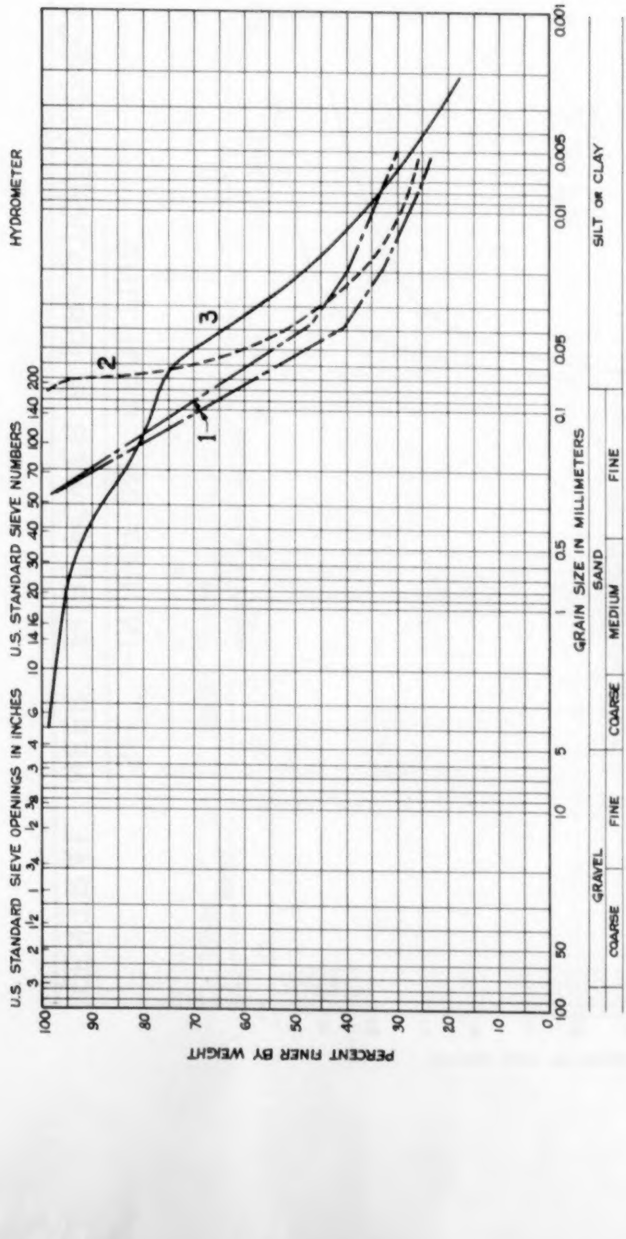
Proportions by Volume				Laboratory 28 day Unconfined Compressive Strength	
Cement	Sand	Bentonite	Water	Lbs/sq. In.	Tons/sq. ft.
1	4	0.35	1.54	414	29.8
			1.34	538	38.8
			1.04	835	60.1
1	8	0.35	2.94	214	15.4
			2.54	242	17.4
			2.14	314	22.6
1	12	0.35	4.14	104	7.5
			3.68	140	10.1
			3.01	195	14.1



1. CAP GROUTING LIMESTONE, C.S. PROCTOR
2. GROUTING FOUNDATIONS & AN EARTH DAM
3. GROUTING EARTH DAM, CORPS OF ENGINEERS
4 & 5. CHICKAMAUGA DAM, T.V.A.

TYPICAL SOILS USED IN SOIL-CEMENT GROUTS

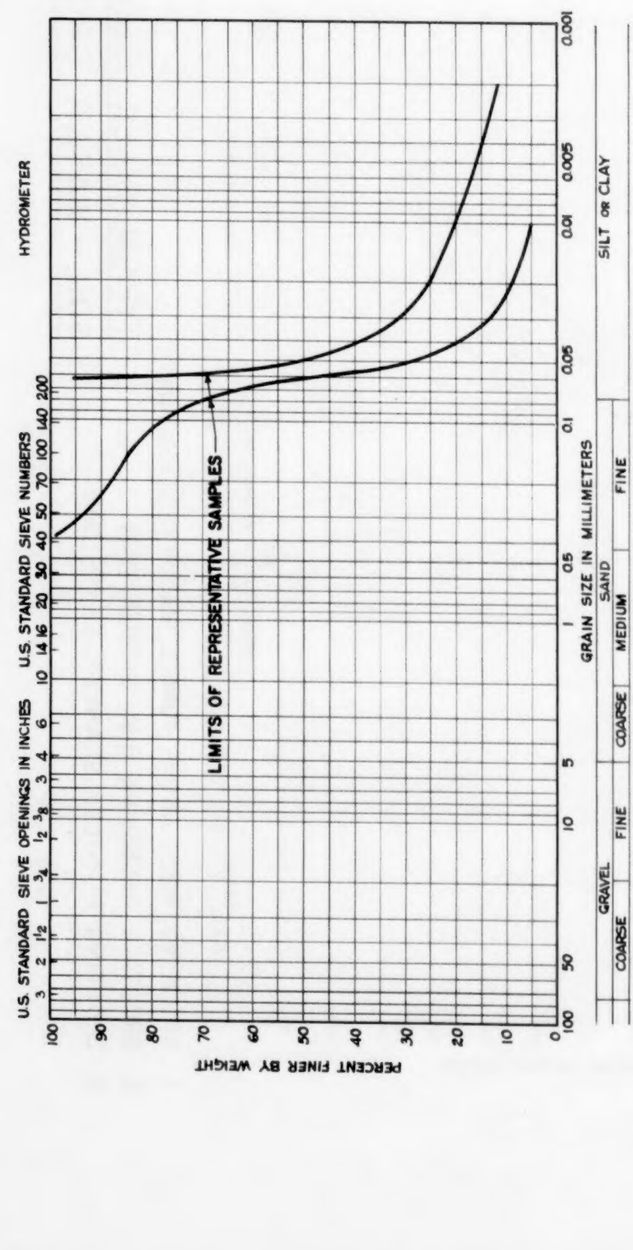
FIG. 1



1. GROUTING VOIDS ABOVE TUNNEL, BUR. OF RECLAMATION
2. GROUTING RESERVOIR RIM, MADDEN DAM
3. GROUTING PERVIOUS BLANKET, EARTH DAM

TYPICAL SOILS USED IN SOIL-WATER GROUTS

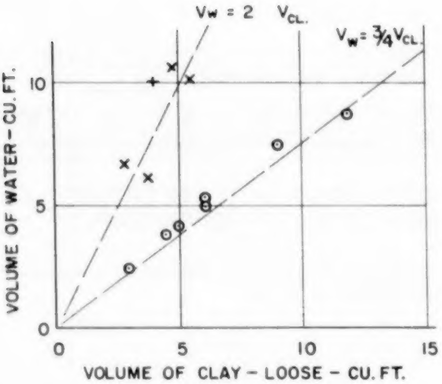
FIG. 2



SILT INJECTION GROUTING

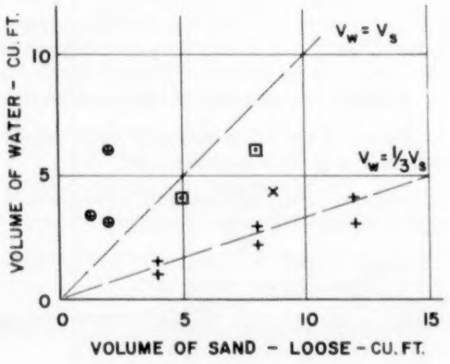
CENTRAL NEBR. PUB. POWER & IRRIGATION DISTRICT

FIG. 3



CLAY-CEMENT GROUT

(FOR 1-BAG OF CEMENT)

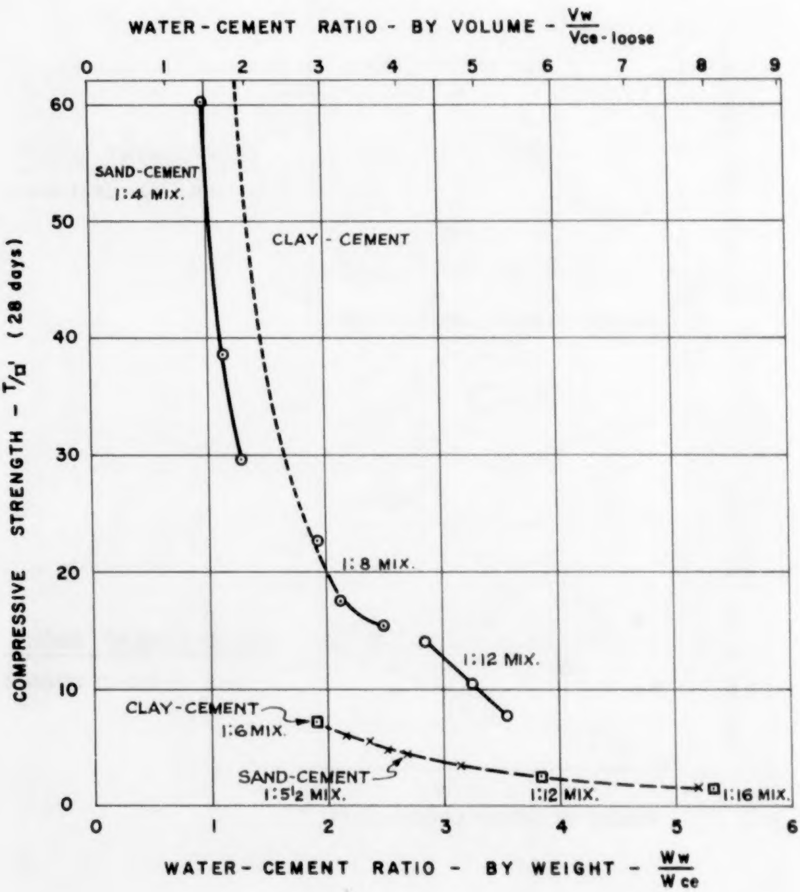


SAND-CEMENT GROUT

(FOR 1-BAG OF CEMENT)

VOLUME OF WATER - SOIL CEMENT GROUTS

FIG. 4



COMPRESSIVE STRENGTH OF CLAY-CEMENT AND SAND-CEMENT GROUTS

FIG. 5

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

CEMENT AND CLAY GROUTING OF FOUNDATIONS: THE USE OF CLAY IN PRESSURE GROUTING

Glebe A. Kravetz,[1] J.M. ASCE
(Proc. Paper 1546)

FOREWORD[a]

The work of the Committee on Grouting of the Soil Mechanics and Foundations Division is divided among four task committees, concerned with (1) bituminous, (2) chemical, (3) cement and (4) clay grouting.

The Task Committee on Chemical Grouting has completed its task and its report was published in the Division Journal, November 1957.

"The Symposium on Cement and Clay Grouting of Foundations" is a "joint venture" of the Task Committee on Cement Grouting and of the Task Committee on Clay Grouting. It includes the following papers:

Proc. Paper 1544 "Cement and Clay Grouting of Foundations: Present Status of Pressure Grouting Foundations" by A. Warren Simonds

Proc. Paper 1545 "Cement and Clay Grouting of Foundations: Grouting with Clay-Cement Grouts by Stanley J. Johnson

Proc. Paper 1546 "Cement and Clay Grouting of Foundations: The Use of Clay in Pressure Grouting" by Glebe A. Kravetz

Proc. Paper 1547 "Cement and Clay Grouting of Foundations: The Use of Admixtures in Cement Grouts" by Alexander Klein and Milos Polivka

Proc. Paper 1548 "Cement and Clay Grouting of Foundations: Suggested Specifications for Pressure Grouting" by Judson P. Elston

Proc. Paper 1549 "Cement and Clay Grouting of Foundations: Pressure Grouting with Packers" by Fred H. Lippold

Note: Discussion open until July 1, 1958. A postponement of this closing date can be obtained by writing to the ASCE Manager of Technical Publications. Paper 1546 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SM 1, February, 1958.

1. Soil and Foundation Engr., Tippetts-Abbett-McCarthy-Stratton, Engrs., New York, N. Y.

a. By Raymond E. Davis, Chairman, Committee on Grouting.

Proc. Paper 1550 "Cement and Clay Grouting of Foundations: French Grouting Practice" by Armand Mayer

Proc. Paper 1551 "Cement and Clay Grouting of Foundations: Practice of the Corps of Engineers" by Edward P. Burwell, Jr.

Proc. Paper 1552 "Cement and Clay Grouting of Foundations: Experience of TVA with Clay-Cement and Related Grouts" by George K. Leonard and Leland M. Grant

These papers are based largely on the results of successful grouting operations for a wide variety of foundation conditions but also are based on the results of laboratory investigations and to some extent on the ideas and opinions of designing engineers who are concerned with foundation problems.

The information contained in the papers has been assembled for the particular benefit of those responsible for the foundation treatment of structures. Recognizing that these papers by no means represent the "last word" in good practice and that there may be many others concerned with foundations who could contribute to our knowledge of cement and clay grouting, discussion is earnestly sought.

ABSTRACT

A brief review of clays and clay-suspensions; of grout mixtures' properties and preparation procedures introduces the study of the following grouts: clay, clay-chemical, clay-cement, and clay-sand-cement. Properties, preparations and testing of these grouts are discussed, as well as some aspects of the field techniques.

INTRODUCTION

Pressure grouting has several applications, one of the most important of these being to reduce the permeability and increase the bearing capacity of a rock or soil foundation. Depending upon the purpose of the grouting and the nature of the foundation to be grouted, different materials such as cement, sand, saw-dust, silt, clay, chemicals, asphalt emulsion, etc., can be used in the grout mixes. Several classifications exist, each giving the types of mixtures that can be successfully grouted into different rock or soil foundations. For classification purposes, the rock foundations are subdivided according to the size of their openings, and the soil foundations according to their grain size distribution. Besides the factors affecting the size of voids, others such as ground water conditions, ph of the soil, etc., must also be considered when selecting grouting materials and methods. Consequently, these classifications are only indicative and when a grouting program is being prepared, soil conditions must be carefully investigated.

In a rock foundation, both wide open faults and haircracks may be found; in a river bed deposit, coarse gravel with free flowing underground water may be interbedded with lenses of fine sand. In a grouting program, therefore, the Engineer should foresee the use of several types of grouts in order to meet

any changes, sometimes very erratic, in the foundation. The range of grouts to be considered for a grouting program are shown in Fig. 1.

It should be noted that for the treatment of rock and granular foundations, clay is one of the grout components. The use of clay, a very common and inexpensive material, not only improves the quality of a basic cement or chemical mix but it also widens the range of its applications. Grouts with clay have extremely interesting properties and their use always betters the final results sought by the grouting treatment. However, through ignorance or misconception, use of clay in pressure grouting can bring very disappointing results or be the cause of serious damage.

For a successful use of clay mixtures, the Engineer must have: (a) a minimum knowledge of the chemical and physical properties of the clay in its natural state and in suspension; (b) a basic understanding of the purposes, possibilities and limitations of any mixture containing clay and a clear comprehension of the clay's role in these mixtures; and (c) a knowledge of a rational and practical way for the preparation of the required grout mixtures. Lack of experience in the application of clay grouts has often prevented their use even though they were the obvious answer to a problem; more expensive or less effective grouts were used instead. When used, their preparation or application followed sometimes empirical methods which did not always bring satisfactory or the best results.

This paper discusses the characteristics of some of the clay containing mixtures used in pressure grouting. Properties of clay and clay suspensions, general grout requirements, laboratory equipment and test procedures are briefly reviewed as an introduction to the following grouts: clay, clay-chemical, clay-cement and clay-sand-cement. The properties and preparation of these grouts are analyzed, as well as the specific grouting methods, when they differ from current rock and soil foundation grouting procedures.

Properties of Clays and Clay Suspensions

Clay Groups

Hydrometer analyses show that clay is a complex compound of minute mineral elements with a predominance of particles smaller than 2 μ (1 micron, μ, is one thousandth millimeter). Chemical and mineralogic analyses show that clay is essentially made of the following elements:

$$SiO_2, Al_2O_3, Fe_2O_3, H_2O$$

and also MgO, CaO, K_2O, N_2O and P_2O_7

Properties of a clay vary according to the intricate combination or grouting of these different elements.

Briefly, it can be said that a clay is essentially composed of acid aluminous-silicic complexes. Its structure is a series of alternating silica and alumina layers of variable thickness. Depending upon the positions of these layers and of the other components, clays are divided into the following groups:

Kaolinite
Montmorillonite
Poligorskite-Attapulgite
Bravaisite-Illite

Of these four groups, the first two are the most commonly found; the third is closely related to the montmorillonite; and the fourth is not too well known. The kaolinite clay is characterized by a silica-alumina ratio varying from 1.5 to 3, and by a compact and rigid structure. The montmorillonite clay has a silica-alumina ratio varying from 3 to 5, and is characterized by a loose structure.(3)

Because of their different type of structure and composition, the physical and chemical properties of these two groups are quite opposite. The kaolinites take very little water and have a low base-exchange capacity. On the other hand, montmorillonites have a high power of hydration and cation adsorption. They are by far the most active clays. The physical properties of the clay in each of these groups may still vary greatly, depending on the nature of the adsorbed elements—Na, Ca, H, etc.—and their concentration. If one of these elements is prevailing, the clay is called accordingly: Na-kaolinite, H-montmorillonite, etc.(11)

Clay Suspensions

The scope of the particle size in the colloidal range is from 5 mμ to 200 mμ (a millimicron, mμ, is one-millionth millimeter). Clay suspensions can, therefore, be considered as colloidal suspensions containing a certain amount of coarser particles.

There are two types of colloidal suspensions, depending upon whether the colloid is a suspensoid or an emulsoid. Clay colloids are of the suspensoid type. In this type of colloid, particles have no apparent affinity for the liquid (lyophobic colloids) although adsorption may exist. The best suspensoid suspensions are obtained in liquids which are good electrolytes, principally in water. The main characteristics of the clay colloidal suspensions are:

1. Each colloid particle carries an electric charge which is usually negative. The charge originates by ionization of the surface minerals or by adsorption of ions from the suspension. According to the Helmholtz concept of the electric double layer surrounding the particles, the inner layer is formed by the ions carrying the charge, while the outer or diffuse layer is formed by the excess of oppositely charged ions in the suspension. The zeta-potential, a measure of the potential difference across the layers, is directly proportional to the charge. If charges are high enough, the force of repulsion maintains the particles in suspension by keeping them in motion (Brownian movement). If charges are very low or are neutralized, the particles collide, then flocculate and settle. Therefore, the higher the charges the more stable will be the suspension. The intensity of particle charges varies with the mineral of the clay, the amount of ions adsorbed per unit area, and the degree of activity of these ions.

On account of its negative charge, each clay particle acts as a group of anions which can adsorb many cations. A clay in its natural state is usually saturated by cations of different elements. If the properties pertaining to a specific element—Na for instance—are sought, these different cations can be replaced by Na cations. One of the many exchange processes can be schematically presented as thus:

$$\begin{array}{l} Ca^{+} \\ Mg^{+}\text{- clay} + HCl\text{, diluted} \\ H^{+} \end{array} \longrightarrow \begin{array}{l} H^{+} \\ H^{+}\text{- clay} \\ H^{+} \end{array} + \begin{array}{l} CaCl \\ MgCl \end{array}$$

$$\begin{array}{l} H^{+} \\ H^{+}\text{- clay} + Na\,OH \\ H^{+} \end{array} \longrightarrow \begin{array}{l} Na^{+} \\ Na^{+}\text{- clay} + H_2O \\ Na^{+} \end{array}$$

2. Clay suspensions cannot be stable. The coarser particles settle eventually under the force of gravity and only the smallest ones remain in suspension. However, the stability of a suspension can be increased through:

a. Mechanical action: Intense initial mixing of a suspension causes the water to wet thoroughly the clay particles and produce a breakdown of possible clay particle aggregates. Further agitation opposes the force of gravity.

b. Electrochemical action: Specific (peptizing) electrolytes in low concentration have a stabilizing effect through the creation of an electric double layer. The stabilizing role strictly belongs to very special ions, which are either formed by the clay itself or closely related to the elements of the clay.

c. Physical action: Viscosity, rigidity and thixotropy can be affected by adding either electrolytes, colloidal material (bentonite) or wetting and dispersing agents.

3. The clay particles in suspension coagulate through the action of added "indifferent" electrolytes which cause the zeta-potential to decrease. Particles are then brought together and adhere because of the Van der Waal's attractive forces. Usually a strong acid or one of its salts is added to a suspension. The metallic ion resulting from the dissociation of this salt has a predominant action in the coagulation. The Hardy-Schulze rule indicates that the higher the valence of the metal the greater the coagulating effect of the added ions. Other factors, however, such as the nature of the added ion, the clay, and the concentration of the suspension, must also be considered. For instance, a diluted suspension may proportionally require more electrolytes than a heavy one. Coagulation by a mixture of two electrolytes may bring variable effects. The combined action of two electrolytes may be greater, equal or smaller than the sum of the action of each of the electrolytes. The rate of the coagulation of a suspension is directly proportional to the coagulating effect of the added ions, the concentration of the suspension, etc. The volume of the coagulated particles varies but an optimum exists which is a function of the coagulant used and its concentration, and the type of clay and its saturating element.

Gelation is a form of coagulation in which the volume of the coagulated particles or gel is equal to the volume of the suspension. The gel is often thixotropic. A smaller concentration of electrolyte is required to obtain it, which indicates that a thixotropic gel is an intermediate stage between stability and flocculation. It appears that the thixotropic state is best obtained with clays containing disc-shaped or rod-shaped particles, such as bentoinite.[5,12]

Summary

The behavior of a clay depends on its chemical composition which in turn governs its structure. A rigid kaolinite structure or a loose montmorillonite one will give the clay, respectively, a lower or higher degree of hydration and base-exchange capacity.

Clay suspensions are not stable; however, their stability and other properties can be modified by changing the saturating element of a clay through the base-exchange process. A strong acid, or one of its salts, will coagulate the suspension and the volume of the coagulated particles will vary with the coagulant and the type of clay.

Grout Mixtures

Properties and Preparation

Grout mixtures can be divided into coarse and fine, according to the nature of their constituent elements. Coarse grouts contain sawdust, rice, fibers and sand with or without a binder material such as cement. Fine grouts contain cement, clay, asphalt emulsion, chemicals or any mixture of two of these materials. With the exception of some chemical grouts and hot tar mixes all the other grouts are suspension systems of some sort. Coarse grouts can be defined as mechanical suspensions, meaning that only mechanical action can maintain them in that state. Fine grouts can be defined as colloidal suspensions and they are divided into suspensiod colloidal (cement, clay, waterglass) or emulsoid colloidal (asphalt emulsion). Stability of colloidal suspensions can be improved through mechanical, electromechanical and physical actions.

All suspensions have similar properties and the problems raised in their preparation are to a certain extent, also similar. Therefore, the Engineer must always check for the following properties when preparing a grout:

1. Stability which is the property that prevents segregation and/or early setting.

2. Controlled setting time to have the grout set at the right place and/or time.

3. Maximum volume of the settled grout to fill a maximum of voids with a minimum of material.

4. Thixotropy and rigidity to prevent washing of the grout by ground water and to help its stability.

5. Fluidity* to facilitate pumping.

Obviously, all grouts do not always possess all these properties simultaneously. When present, their relative importance may vary with the type of grout or job requirements. Nevertheless, any grout mixture, when studied, should always be tested for each of the above mentioned properties.

After grouts have been tentatively selected corresponding materials (clay, sand, cement, chemicals) are sent to a soil laboratory. In the case of a soil

---

* Fluidity: unit Rhe, dimensions cm^2/sec-dyne ($M^{-1}LT$) as opposed to
Viscosity: unit Poise, dimensions dyne - sec/cm^2 ($ML^{-1}T^{-1}$)

foundation, samples of the foundation material and ground water are sent also. Ground water is analyzed for its ph and salt content. Soil samples are tested for grain size distribution and relative density. Grout materials require the following tests:

1. Sands: grain size, clay or organic matter content.
2. Cements: fineness, setting time for water-cement ratio equal or higher than 10, water gain for water-cement ratio higher than 0.5.
3. Clays: moisture content, Atterberg limits, hydrometer analysis, sand and/or organic matter content.
4. Chemicals: concentration, impurities, etc.

Grout mixtures are then prepared with the selected materials. As already mentioned, the properties of the grout sought vary with the job conditions and their composition is determined accordingly. Groutability of the mixture is then checked by field grouting tests. However, grouts for soil foundation treatment are checked during their preparation by laboratory grouting tests.

Laboratory equipment and most of the preparation procedures being identical for all these grouts, a general discussion of them follows below. Detailed preparation procedure for each of the different grouts studied in this paper will be presented hereafter.

Laboratory Equipment and Procedures

Most of the laboratory equipment required for the study of grout mixtures are borrowed from soil and materials laboratories or from oil fields mud-testing laboratories. The equipment for the preparation and testing of grout mixtures includes the hydrometer test apparatus, standard glassware (replaced by waxed paper containers for cement and other hard setting grouts), viscosimeters and a pressure grouting device.

The general procedure for obtaining the required grout will be as follows: the constituent elements are mixed in varying proportions and the resulting grouts are tested for viscosity, rigidity and thixotropy, stability, flocculation, groutability, permeability, and strength. Enough grout must be prepared to run the tests concurrently, especially if the grout settles into an irreversible form. If clay is available only in small quantities, the same grout can be reused after each test, provided it is mixed again.

Mixing Procedure

Proper mixing of the grout is extremely important, both in laboratory and in the field. Mixes should always be prepared by adding the different solid materials in their order of fineness. Water is always poured first; reagents, for obvious reasons, are mixed last, especially if quick setting grouts are prepared. The grout must be stirred when adding a new material. Materials should always be poured slowly into a mix. This is particularly important for clay and reagent. Material already in the mix must be thoroughly dispersed and wetted before adding the next one. Laboratory mixing is usually done with high speed mixers, however, in the field, standard grout mixers are usually much slower. When preparting the grouts in a laboratory, the Engineer should, therefore, take into consideration the type of equipment to be used in the job and design the mixes accordingly.

Stability and Flocculation Tests

The stability of a grout is most easily studied on diluted suspensions having a water-solids ratio of 10 or more. Several cylinders are filled with a suspension, then increasing quantities of each of the stabilizing agents tested are added to all but one. From thereon, the standard procedure for a hydrometer test is followed. Results can be expressed either by the usual effective diameter size curves or by an effective diameter size versus time curves. By comparing the different curves obtained, optimum proportions of each stabilizing agent are determined (usually in percentage of the weight of the grout material). This method is not applicable to rigid or thixotropic grouts nor to dense mixes. A thixotropic grout is normally stable enough for grouting purposes. When required, rigid and dense grouts are stabilized by using the optimum proportions determined from dilute suspensions, although it is known that proportionally less stabilizing agent is required for a denser mix.

The flocculation test is the same as the stability test but a flocculating agent is added to the stabilized suspension. Results may be expressed again by the effective diameter size curves or by an effective diameter size versus time curves. Usually only approximate time of beginning and end of flocculation is noted and, therefore, grouts of any rigidity or density can be tested. At the end of the test the volume V_c of the coagulated material is measured and compared to the original volume of the grout V_s. The ratio V_c/V_s passes by a maximum when added quantities of the flocculent increase. The amount of the flocculent (expressed in percentage of the dry weight of the grout material) that gives the highest V_c/V_s ratio is adopted. If dense grouts are found to have a V_c/V_s ratio of 1, the amount of stabilizing agent is decreased and proportions of stabilizer readjusted until a V_c/V_s ratio under 1 is obtained.

Viscosity, Rigidity and Thixotropy Tests

The two most commonly used methods for measuring the viscosity of a grout are the Marsh and the Stormer type viscosimeter tests. The Marsh type viscosimeter is generally used. It is a cone-shaped funnel 6 inches in diameter at the top and 13 inches high. The discharge tube at the bottom is 3/16 inch inside diameter. A No. 8 mesh screen is fitted across half the cone, 3/4 inch below the top lid, for the purpose of removing any lumps from the grout. The other half remains open to facilitate cleaning.

There is no standard procedures for the test. Methods differ in the volume originally placed in the funnel, and the volume timed as it flows out. The method used, however, must be the same throughout a study, and should be indicated with the test results. Marsh viscosimeters are sometimes altered in order to have interchangeable discharge tubes. For extremely dense grouts with high rigidity no discharge flow can be obtained even from the biggest discharge tube. This can be overcome either by adapting a small vibrator to the funnel or by fixing it on a shaking table.

The Stormer type viscosimeter is used to measure quantitatively the viscosity, rigidity and thixotropy. It consists principally of a spindle or bob activated by gears driven by a falling weight which can be adjusted so that the spindle may rotate at any desired speed. Viscosity is determined as a function of the weight which makes the spindle rotate at a fixed speed, usually 600 rpm.[2]

Quantitative values of the rigidity, defined also as yield value, and thixotropy can be determined by plotting curves of rpm versus weight. Such a typical curve is given in Fig. 2. The writer does not know, however, of this method having ever been used in grout preparation although it would be of unquestionable interest. Thixotropy is usually determined qualitatively by noting the time for which a grout would no longer flow from an inverted cylinder. Rigidity is measured with a hydrometer. The grout's rigidity does not permit correct hydrometer density readings, as they show a higher value than the true one. Consequently, the rigidity of the grout can be expressed by the following relation:

$$R = A\,\frac{d_1 - d_o}{d_o}$$

where

- d_1 is the grout density read on the hydrometer
- d_o is the true grout density as measured by weight
- A is the ratio of the hydrometer weight to the lateral surface of its portion submerged in a grout of apparent density d_1. A must be established for each hydrometer used. It is a direct function of d_1.

The above procedure indicates, incidentally, that it is better not to measure the density of a grout with a hydrometer.

Grouting Tests

All grouting test equipment use the same method: controlled pressure is applied into a tank containing the grout, which is thus forced into another reservoir in which has been placed the soil material to be grouted. The soil containers may be of various types, such as a plywood box with removable sides; a glass tank for watching the progress of the grout; or a cylinder from which the grouted material can be removed with a soil sample extruder. The grout itself may be forced into the soil tank from the top or from the bottom, or through a perforated pipe placed at the center. In order to imitate natural conditions the granular soil is placed both in loose and dense condition and the tank is then gradually filled with water. This can be done in one single test or in two successive ones. Before testing is started the grouting device should be checked by injecting dyed water into the sand. During the test, water expelled from the tank is measured in order to check the progress of the grout. Because of the relatively small quantity of soil grouted, this test imitates the grouting of shallow holes in which the grout always tend to come to the surface of the ground rather than penetrate the soil. The same thing happens in a tank with soil where the grout tends to find the shortest way to the outlet. This can be somewhat corrected by having several outlets at different locations, alternately opened and closed.

Grouting tests, in spite of their ingenious procedures and equipment, are not very satisfactory. The Engineer cannot ascertain that the soil, as placed in the tank, represents the natural conditions in the foundation. Only low pressure can usually be used and homogeneous penetration of the soil cannot

always be obtained. Therefore, analysis and interpretation of the results are very important. Grouted soils are sliced or broken to study the penetration of the grout, and samples are cut for permeability and compressive strength tests.

Permeability and Compressive Strength Tests

These tests are made on samples of the natural soil, the grout itself, the grouted soil and an "optimum grouted" soil sample. Procedures for these tests are standard and will not be described here. However, the following points are of importance:

If the grouted soil is always below ground water level, the samples tested should be kept in water permanently. For instance, clay grouts when tested for permeability will be left to set in the permeameter. Samples of clay-cement and other grouts, or samples of grouted soils will not be allowed to dry before "curing." It might be found that some of them will gradually disintegrate during the "curing" period, while if exposed to air, they could reach unusually high compressive strength values.

The "optimum grouted" sample is prepared by pouring the soil into the grout, then if required, vibrating the mix to a higher density. Density, permeability, and strength tests on this sample would indicate the optimum results which could be obtained under perfect grouting condition. The grout required to fill 100% of voids could also be computed. These results would show, therefore, the maximum improvements of the soil obtainable with the selected grout. By comparing these results with the best results of laboratory grouted samples, the efficiency of the grout is determined.

Clay Grout

Grout Requirements

The first and obvious requirement for the selection of a grout is that its particles should be smaller than the voids to be filled. This may be determined by the following equation:

$$\frac{D_{15}}{D_{85}} > N$$

where

D_{15} is the 15 per cent size of the soil

D_{85} is the 85 per cent size of the grout material

N is a dimensionless number

Fig. 3 is a graphical interpretation of the above equation. It shows:

a) Typical grain size curves for Portland cement, Boston blue clay, ordinary asphalt emulsion, and special Shellperm asphalt emulsion; and

b) the lower limits of sand groutable by each of the above grout material, as indicated in Fig. 1.

The value of N obtained by comparing these curves is 15.4 for the clay.

However, N values ranging from 5 to 20 have been proposed for clay grouts, depending on local condition.

The second requirement is the stability of the grout. Sometimes, sand foundations cannot be grouted by cement mixture, not because of the particle to void ratio requirement, but because of the low stability of the cement mixtures. Their fast flocculation and the resulting plugging of the voids prevent further penetration of the grout. The success of clay grouts is due to their good stability during and after the grouting operations. Stability during the grouting depends not only on what could be called the "intrinsic" stability of a grout, but also on its thixotropy and rigidity. The intrinsic stability of a grout is its ability to retain the homogeneity for a predetermined amount of time. This is obtained through the use of stabilizers, which also affect its fluidity. Rigidity and thixotropy give better resistance to the grout against water and allow it to resume its flowing if pumping was interrupted.

Stability after the grouting depends also and essentially on the rigidity and thixotropy of the flocculated grout. All grouts eventually flocculate. If only gravity forces are acting, the coarser particles settle first, and although in their downward movement they may collide with finer elements, the resulting deposit is definitely segregated. Segregated deposits have a low V_c/V_s ratio, very little rigidity and are not thixotropic. When flocculents are added to a suspension, floccules start to form and at the beginning their distribution is uniform through the volume of the suspension. Each floccule, therefore, is made by the agglomeration of grains of all sizes, and the final deposit is homogeneous, or at least, representative of the suspension condition when flocculation started. Such a flocculated deposit has a high V_c/V_s ratio and is rigid. This is why the third requirement is that flocculents must be added to the grout; not to produce a flocculation, which would occur anyhow, but to give the flocculated grout its special properties. Thixotropy of a grout depends on the clay type and on the ability of the grout to flocculate into a gel whose V_c/V_s ratio is equal to 1. The opposite, however, is not true. Not all grouts with a V_c/V_s ratio of 1 are thixotropic. Experience shows that the V_c/V_s ratio is directly proportional to the density of a grout and so is the rigidity.

The last requirement is that the grout be pumpable. Whatever the other properties of a grout might be, it has to be fluid enough to flow through the voids of the ground or, for heavy mixtures, to be pushed by pumps. Fluidity of a grout, however, is inversely proportional to its density.

Thus, it can be seen that the different properties sought in a clay grout are closely connected and cannot be improved or developed independently.

Grouting Program Study

In preparing a grouting program which includes the use of clay mixtures, the Engineer is generally restricted both in time and money. A comprehensive study should include (a) the determination of a clay's mineral elements through X-ray examination, adsorption tests and differential thermal analysis; (b) the evaluation of a clay's properties variations through different (Na, K, Li, Rb, Ca, Mg, H, etc.) saturating cations; and (c) the determination of the influence which stabilizers and flocculents have on each saturated clay. But in the field, the Engineer's purpose is only to determine a group of workable, practical and economical grouts. His study must, therefore, be limited in scope.

Clay Sample Selection

The first step is to make sieve analyses of typical soil samples and hydrometer analyses of the different clays available, and then have the grain size curves checked against the grout requirements (Fig. 1) and the soil-grout material equation (Page 18). In performing the hydrometer analyses, the procedure for mixing the clay at the job must be kept in mind: if the clay is dried first, ground and then mixed, the hydrometer analyses should be made on dried samples. If, however, the clay is not allowed to dry but is immediately placed in water tanks and gradually diluted, the hydrometer analyses should be made on similarly treated samples. This is extremely important. It is known that some clays when dried even partially, agglomerate to form small aggregates that the most thorough grinding, wetting and mixing will not break. For such clays, therefore, field methods of mixing could be the difference between a successful or unsuccessful grouting. Conversely, hydrometer analyses made on both dried and moist preserved samples, may indicate what type of field mixing will be acceptable.

Concurrently with the hydrometer analyses of the clay samples, suspensions are prepared with a clay-water ratio of 0.5. Following a thorough mixing they are allowed to set for 48 hours, after which time their respective V_c/V_s ratio is measured. All grain size curves and results of the sedimentation are compared. Sometimes, for instance, it may be found that a sandy clay has a high V_c/V_s ratio, and could, therefore, be used very successfully if the sand material can be eliminated. In any case, following these two tests one sample is usually selected for the preparation of the grout.

Stabilizing and Flocculating Agents Selection

It has already been mentioned that the physical properties of a clay may change greatly depending on the nature of the adsorbed cations. For example, the stability of a clay suspension decreases in the following order

$$Li > Na > K > Rb > Ca$$

while the swelling of bentonite takes the following

$$Na \geqslant Li > K > Ca = Ba > H$$

The viscosity, hydration and other properties vary similarly.(3) Because the order of intensity of the different properties varies, there is no single element which could give optimum qualities to the grout. It has been found, however, that Na, K and Li-clays possess the most satisfactory qualities. This is the reason why the following electrolytes are commonly used to stabilize (peptize) clay grouts:

Potassium Nitrate	KNO_3
Potassium Carbonate	K_2CO_3
Sodium Aluminate	$NaAlO_2$
Sodium Silicate	Na_2SiO_3
Lithium Carbonate	Li_2CO_3
Sodium Hydroxide	$NaOH$

Quantities used are very small and are expressed either in grams per liter of grout (average 1 to 10 g/1) or preferably as a percentage of the clay weight (.25 to 5%).

Electrolytes are not the only material used to stabilize clay grouts; bentonite is also used. In very small concentration it makes extremely stable and thioxotropic grouts which are ideal for sand foundation grouting. Unfortunately, bentonite is expensive. However, when added in small quantities to a grout (1 to 10% of the clay weight), it gives stability, rigidity and thixotropy. For light grouts (water/clay = 10), it is very often the only effective stabilizer. Therefore, when bentonite is available, it is very often only a matter of economic consideration to decide whether it should be used. Wetting or dispersing agents may be successfully used to increase the stability of a grout and decrease its viscosity.

Electrolytes used as flocculents are usually salts of the sulphuric and hydrochloric acids. The most commonly used are:

Aluminum Sulphate	$Al_2(SO_4)_3$
Sodium Sulphate	Na_2SO_4
Calcium Chloride	$CaCl_2$
Copper Sulphate	$CuSO_4$
Ferrous Sulphate	$FeSO_4$

When sodium silicate is the stabilizer, HCl, $NaAlO_2$ and other chemical grouting reagents are used. This list, like the preceding one, is not complete as other electrolytes are available.

Flocculents are used in even smaller quantities than stabilizers; proportions are expressed in grams per liter of grout or in percentage of the clay weight.

The recommended procedure for cation exchange is to treat the clay with H_2O_2 in order to remove any organic matter, wash it with diluted HCl, and then treat the clay with the selected electrolyte. This, however, is not done in the preparation of a clay grout. The clay suspension is stabilized directly by each of the stabilizers tested, and optimum proportions are determined as described above. The test procedure does not change if stabilizers other than electrolytes are used. The stability of a suspension usually increases with increasing amount of stabilizer; with some electrolytes, results are sometimes very erratic and an optimum may exist past which additional stabilizer is detrimental.

The action of a flocculent depends on the stabilizer used in the suspension. Therefore, each flocculent should be tested with differently stabilized grouts. This may lead, in practice, to a considerable amount of testing, which very often cannot be afforded. Results of the flocculation tests show that (a) for a stabilized grout with a specific density the volume V_c passes usually be a maximum when flocculent proportions are increased, and (b) for increasing density, the volume V_c increases also, but the ratio V_c/V_{co}, in which V_{co} is the volume of the flocculated material when no flocculent has been added, decreases gradually. This indicates that above a certain clay-water ratio (approx. c/W = 2), the proportions of a stabilizer giving a maximum V_c are fairly constant.

Viscosity, Rigidity and Thixotropy

Each grout prepared for the flocculation test is checked as follows:

Viscosity is measured and thixotropy is observed at any convenient time. Rigidity is measured by the hydrometer method immediately before and after the flocculation test.

Grouts used for soil foundation treatment must have a very low viscosity. Viscosity like rigidity is inversely proportional to the water-clay ratio: this is why grouts used should have usually a water-clay ratio higher than 3.

Rigidity and thixotropy cannot be improved by adding more of the basic clay constituents as viscosity would also increase. It is usually done by adding bentonite (1 to 10 g/l) or by using sodium silicate with a reagent. Very small amounts of sodium silicate (for instance, Water-clay = 8, Water glass = 15 g/l, $CaCl_2$ = 2 g/l) are used so as to have the grout retain all the characteristics of a clay grout. Nevertheless, small floccules form in the mixture which stay in suspension and give the grout rigidity and thixotropy. If the grout is stirred several times, it loses its thixotropy.

Laboratory Grouting Test

If the requirements of pressure grouts use (Fig. 1) and the soil-grout material equation (Page 10) are met, the grouting will establish the maximum acceptable viscosity. Sometimes, it corresponds to such low clay-water ratio that grouting is no longer practical as the volume of grout to be pumped is disproportionate with the results obtained. Clay-chemical or chemical grouts must then be anticipated, though only field grouting tests can definitely establish the suitability of the grout for injection.

Permeability Test

Hydraulic, economic or safety requirements may impose a maximum value for water seepage through a dam foundation. The ratio of the coefficient of permeability before grouting, Kn, to the coefficient of permeability after grouting, Kg, expresses the improvement required. Comparison of Kg to Kn, as obtained by laboratory grouting, and Kgo, as obtained from the "optimum grouted" samples, indicate the maximum improvement which could be obtained (kn/Kgo), the minimum (Kn/Kg), and the efficiency of the grouting (Kg/Kgo). It must be borne in mind that laboratory grouting results are usually on the safe side and that success of a grouting program depends also on the pattern of grout holes, the grouting sequences and methods, and the combination of grouts used.

Clay Chemical Grout

Until recent years, sodium silicate was the only chemical used in pressure grouting. Later, the chrome-lignin process was developed and patented, and more recently, a variety of chemical injection processes have been studied by T. W. Lambe.[6] Most probably clay could be mixed with many of these chemicals to form clay-chemical grouts. However, it is believed that clay has only been used with water glass, therefore, only clay-water glass grouts will be discussed.

It has already been pointed out that clay grouts, to which water glass was added to reduce the viscosity and increase the thixotropy, are different from

clay-chemical grouts. In the clay-grout, water glass is used as an additive in very small proportions and thus the grout retains its basic properties. In the chemical-clay grouts, clay is used as an additive and, although it gives new qualities, the mixture retains all the properties of a chemical mix. The principles of the clay-chemical grouting and chemical grouting are, therefore, identical.

Chemical Grouting

Chemical grouting was first developed by Albert Francois. Since then, different methods and techniques have been developed and several processes established, among which are those of Joosten, Gayard, Langer and Riedel. All of them follow one of two basic methods.

In one method, water glass and reagent are injected in the ground separately through alternately spaced pipes. Contact of the two solutions occurs in the ground and the flocculation is practically instantaneous.

In the second method, water glass and reagent are mixed first in pre-established proportions, then pumped into the ground. Proportions of the mixture can be determined in such a way that any desirable setting time in a range varying from a few minutes to several hours can be obtained. This method is usually preferred because it is more flexible, easily adjustable to variable conditions and more economical. The first method is used in emergency work where immediate solidification or water seepage control is required.

Both methods are based on the same principle. Water glass treated by an electrolyte gives lyophobic colloidal solutions which eventually flocculates and settles into an irreversible rigid gel.

In the first method, the formation of the gel is practically instantaneous because both the water glass and the electrolyte are highly concentrated. Gel obtained by this method is comparatively rigid.

In the second method, setting time can be controlled by diluting the water glass or reagent, or both. Rigidity of the resulting gel decreases as the dilution increases. 40° Baume sodium silicate is used and is usually diluted with an equal volume of water.

Reagents most commonly used are: hydrochloric acid, (HCl), sodium aluminate ($NaAlO_2$), and calcium chloride ($CaCl_2$). Two reagents may be used instead of one. The second reagent should be a salt from a strong acid (SO_4CU, SO_4Fe, etc.). The concentration of the reagent solution which will give a certain setting time to the chemical grout varies with the electrolyte used, the purity, quality, titre of the product, etc. Most of the time it is determined from practical experience.

Fig. 4 represents typical curves which give the setting time of a 50-50 solution of sodium silicate and water versus the amount of the reagent solution added. These curves were established on altogether different jobs or studies and totally different products were used. For instance, for curve #3, $NaAlO_2$ was obtained as a concentrated solution, while for curves #4 and #5, it was a white-grayish powder. The products for curves #4 and #5 were from different manufacturers. The curves indicate the beginning of the set as the first floccules start to form in the transparent mixtures. Total set occurs in 5 to 30 minutes, depending on the dilution of the products.

For all these curves variable quantities of predetermined reagent solution were added to a fixed quantity of water glass solution. Consequently, for each

point of a curve the reagent-water glass and water-water glass ratios are different.

Another approach is to determine and "optimum" reagent-water glass ratio (usually giving a ph = 7 to the solution) and then make the water ratio vary. The mixing procedure, however, becomes more elaborate.

Mixing procedures are extremely important in chemical grouting. Figure 5, which is true not only for chemical grouting but also for quick setting mixtures of Portland and Alumina cements, shows that for increasing quantities of reagent the setting time decreases first, is instantaneous for a certain range, and then increases. Only grouts belonging to the left part of the curve are used for chemical grouting. Figure 5 also shows that for these types of grouts, the reagent must be added to the water glass solution as otherwise, their mixture could have, during mixing, the proportion giving an instantaneous set.

Preparation of a Clay Mixture

Very often, clay-chemical grouts are used in a grouting program together with clay grouts and chemical grouts. If such is the case, the clay grout, as prepared, is tried for the clay-chemical grout. If a clay mixture has to be prepared specificatlly for the clay-chemical grout, its preparation will be similar to the one described under the heading Clay Grout. Because the sodium silicate acts as a stabilizer and is more sensitive than clay suspensions to electrolytes, bentonite or water glass are used preferably as stabilizing agents.

Selection of the Chemicals

Even if a chemical grout is not used on the job, a few trial solutions of water glass and selected reagent are prepared in order to check their behavior. The water glass is first diluted with an equal volume of water. Proportions to which the reagent is diluted will vary as shown in Fig. 4. Strong reagent solutions make the setting more difficult to control and mixing operations more hazardous. Extremely diluted solutions will give only slow setting times and low-rigidity gels. With an adequate dilution of the reagent, the setting time versus reagent curve should have little or no dispersed points, and cover a predetermined range of setting time (varying with the job condition) with no difficulties.

Preparation of the Clay-Chemical Grout

In preparing the clay-chemical grout, it should be remembered that: (a) extremely short setting time can be established during preparatory study, however, on the job the grout must be mixed, pumped, and made to penetrate the soil before it can set, therefore, the minimum setting time is dictated by the field conditions (usually half an hour); (b) there is a minimum amount of sodium silicate below which the setting time of the clay-chemical grout cannot be controlled although the gel will form eventually, this amount varying with the clay proportions, the reagent used, the water ratio, etc. However, it corresponds approximately to a final water glass/water ratio of 0.25 to 0.30.

Preparation of the grout is done in the following manner: A dense clay grout is prepared, undiluted sodium silicate is added in a proportion of 250cc for 750cc of the clay mixture, increasing quantities of reagents are added,

and setting time established for each separate mixture. During these three operations, the following things may happen: (a) when adding the sodium silicate to the clay mixture some floccules form in the mix, although total flocculation does not occur; (b) after the reagent has been added, the clay flocculates and settles before the sodium silicate forms into a gel.

Spot flocculation of the sodium silicate may be caused by some of the free anions in the clay suspension. This can be corrected by taking the following steps: add the sodium silicate slowly; or add the sodium silicate diluted; or do not use electrolytes to stabilize the clay (this is the most usual and effective remedy). Flocculation of the clay can be prevented by: stabilizing the clay grout preferably with bentonite, or using a higher sodium silicate-water ratio for the clay-chemical grout.

Several grouts have to be prepared in order to meet any change in the soil foundation. Preparation of a thinner grout will be done in the same manner. Starting from the same heavy grout water will be added, then sodium silicate, then reagents, and setting time versus reagent curves established.

Two clay grouts (curves #6 and #7) are shown in Fig. 4. It seems that in the clay-chemical grouts, part of the water is immobilized by the clay and does not influence the water glass-reagent reactions. This can be clearly seen by considering curve #6. The 100cc of the clay-chemical grout represented by this curve could be considered as being made of two different grouts; 50cc of a 50-50 water-water glass solution, and 50cc of a dense clay grout having a water-clay ratio of 1.25. It can be further assumed that the reagent with the 100cc of the clay-chemical grout reacts only with the 50cc of the 50-50 water-water glass solution. This assumption is illustrated by curve #8 which shows the amount of reagent required for 100cc of a 50-50 water-water glass solution. Therefore, it can be compared to curves # 1, # 2, #3, #4 and #5. Similarity of curves #5 and #8 seems to justify the assumption made. This suggests that when the clay-chemical grout sets, the coagulated water glass acts as a lattice which encloses the clay grains and a certain amount of free water. In any case, the comparison of curve # 6 with curves #3, #4 and #5 is extremely interesting as it shows that by adding clay to chemical mixtures, grouts of equivalent qualities can be obtained using less chemical products (50% less in the above example.)

Other Tests

Following the preparation of the different grouts, grouting tests are run to check the maximum acceptable viscosity of the grouts. The reduction of permeability and/or the increase of strength of the soil is then tested according to the above mentioned testing procedures. If necessary, the grouts are corrected for: (a) better penetration or lower viscosity by reducing the clay content; and (b) better impermeabilization and strengthening by increasing the sodium silicate content.

Clay-Cement Grout

Of all the mixtures containing clay, the clay-cement grout is probably the best known and the most commonly employed. For soil foundation, the cement is used as an additive in proportions usually enough to give the grout a cement-like set. This type of clay-cement grout has, therefore, all the basic properties of a clay grout. In rock foundation, the cement proportions can be

much higher, however, even grouts with a very high cement content have, because of the clay, very distinct qualities.

Cement Grouting

Standard cement grouts are unstable, have a low viscosity and no rigidity for water-cement ratios (in weight) higher than 0.5. For lower values of the water-cement ratio, the viscosity increases sharply and the grout acquires some rigidity. For a given cement, its density after setting is fairly constant and independent of the water-cement ratio of the grout. The density varies with the type of cement used: the volume of hardened cement grouts which have identical proportions and are mixed under similar conditions may vary as much as 50%. All standard grouts set with a water gain. Their setting time increases when the water-cement ratio increases. Some cements never set for water-cement ratio higher than 10.

Cement grouts can be improved by mechanical or physico-chemical action. High speed mixing (1500 to 3000 r.p.m.) activates the hydration of cement particles, gives a better dispersion and brings a rapid formation of small crystallized elements of hydrates of different types. Groouts obtained through energetic mixing are sometimes called "activated" or "colloidal." Equipment and methods for their preparation have been patented and trade names registered (Colcrete, Colgrout, High-turbulence). Basically, these grouts are more stable and more fluid than ordinary grouts but only for very low water contents.

Wetting or dispersing agents also improve the dispersion and hydration of the cement particles. Resulting grouts are found to be more stable, and more fluid than ordinary grouts with the same water content. Engineers have been reluctant to use wetting or dispersing agents for ordinary concrete, although these are used in the concrete and grouts made by the "Prepakt" method.

Clay-Cement Grouts Properties

The first clay-cement grouts ever used were of two radically opposite types. First, bentonite was added to cement grouts in very small amounts (1 to 3% of the cement weight) as it was noticed that it was improving the stability of the mixture without affecting appreciably its compressive strength. Simultaneously, use of cement was suggested to give a hard set to clay grouts. In the meantime, it was felt that excessive amounts of cement were sometimes used unnecessarily in pressure grouting and use of clay-cement grouts was proposed as being more economical. It is interesting to note that the main concern in the first applications of clay-cement grouts was to limit the drop of compressive strength of the grout when its clay proportions were increased.(9)

Since then, it has been gradually realized that compressive strength of cement or clay-cement grouts is of minor significance, both for impermeabilization and consolidation grouting, and that anyway, there is very little if any relation, between the strength of some cubes of grout prepared and cured in a laboratory, and the strength of the same grout after it was pumped into rock cracks or openings. With emphasis on the grout strength gone, more attention has been paid to the fundamental qualities of clay-cement grouts, which are:

1. Stability: It is directly proportional to the quality of the clay and its

proportion in the grout. A small amount of clay is usually required to obtain grouts which settle with little or no wate: gain. Grouts with no water gain are always sought whenever possible since with them there is no need for redrilling and regrouting to complete the filling of voids. They are a must in grounds where excess water may be detrimental (contact grouting behind tunnel linings in rock or soil strata, for instance).

2. Rigidity and thixotropy: Grouts which are made to settle with no water gain can be thixotropic sometimes, that is, until the cement hardens. Clay-cement grouts always have some rigidity.

3. Adherence: This is a very interesting property of the clay-cement grouts which are found to have a much better adherence to clay-coated walls of cracks and faults in rocks, than cement grouts.

4. Economy: This is the most important of the qualities of clay-cement grout as shown by the following example: filling of one cubic yard of voids by cement grout requires an average of 1000 lbs. of cement; the same volume could be filled by clay-cement grout made with 100 lbs. of clay and 300 lbs. of cement with a .75 solids-water ratio.

Preparation of clay-cement grouts for a soil or rock foundation is basically the same. For soils, the clay-cement ratio of the grout is usually fairly constant simplifying greatly the grout preparation. For rocks, however, grouts with different clay-cement ratios are usually anticipated, and to be economical their study must be systematic.

Study and Preparation for Soil Foundation

When preparing a clay-cement grout for a soil foundation treatment, the procedure should be as follows:

1. Preparation of a clay mixture: The method is identical to the one described in the section on Clay Grout. When there are no predetermined proportions of the tentative clay-cement grout, a heavy mixture is usually prepared.

2. Adding of Cement: The proportions of cement must be determined so that (a) the settled grout will eventually harden; (b) the V_c/V_s ratio is acceptable when the cement starts its setting. Condition (a) corresponds to grouts with stable clay suspensions: it gives the minimum amount of cement that can be used. This amount varies with the cement and/or clay used and the solids-water ratio. The usual proportions for a trial mix are 100 grams of cement for 1 liter of clay-grout. Condition (b) corresponds to grouts made with unstable clay-mixtures. Grouts used for soil foundation treatment have always a low viscosity, therefore, a low solids-water ratio sometimes makes it difficult to obtain stable grouts. After the cement has been added, the grout is diluted to a satisfactory solids-water ratio and then checked for stability, flocculation and viscosity.

3. Stabilization of cement grouts: At times, it might be necessary to stabilize the grout after the cement has been added. Stabilizers and procedures used are the same as those for clay grouts.

4. Quick-setting grouts: They are required when there is a strong ground water circulation or if grouting must be limited to a restricted volume of soil. Quick-setting can be obtained by: (a) adding Calcium Chloride;

(b) mixing Portland and Alumina cements; or (c) adding Sodium Silicate and reagent. Methods (a) and (b) are standard procedures in cement grouting. When used with a clay-cement grout having a high clay content their efficiency is limited and method (c) is adopted. By this method, the grout sets into a silica gel on account of the water glass and reagents and is maintained in a homogeneous state until the cement starts to harden.

5. Grouting tests: Because of the grain size of the material to be grouted, the laboratory tests are more difficult; grouts tend to find the shortest way to the outlet very easily, and bigger tanks should be used to obtain better results.

Study and Preparation for Rock Foundations

For rock foundation grouting, the grout described above may be used for the filling of very small cavities. The purpose of the clay-cement grouts, however, is to replace ordinary grouts. Therefore, grouts will be used with solids-water ratios ranging from 0.1 to 2.0. Field preparation and changes from one grout to another should be simple operations. Nevertheless, the quality of the grout should always be under control. Preliminary study of clay-cement grouts for rock foundation grouting consist essentially in establishing a set of curves as shown in Fig. 6. This figure gives the following basic curves:

1. A series of viscosity versus consistency curves. Viscosity is expressed in centipoises, Stormer viscosimeter, at 600 r.p.m. Consistency is expressed by the ratio of the two solid materials (clay and cement) to water by weight. Shown are curves of a treated clay grout (#0), cement grout (#10), and clay-cement grouts of different clay-cement ratios (#5 to # 9). Four grouts of different densities (giving four points on each curve) are usually enough to determine curves #0 and #10. Because of similar shape, each of the curves # 5 through # 9 do not require more than one or two points.

2. The lines of equal V_c/V_s ratio with V_c/V_s varying from 0.6 to 1. Experience shows that points of equal V_c/V_s ratio for the different grouts are located on a curve which can be assimilated to a straight line if values of grouts #0 and #10 are disregarded. This is particularly true for values of V_c/V_s higher than 0.8. For a preliminary study it is, therefore, enough to determine the V_c/V_s value on any two of curves # 5 through #9.

The figure as established shows graphically the basic properties of all obtainable grouts. It can be completed by adding curves of equal density, compressive strength, rigidity, setting time, etc., if the scope of the grouting program required it.

If the minimum values of V_c/V_s and maximum values of viscosity are determined, all acceptable grouts will be located within an area such as b-c-d-e (Fig. 6). Grouts falling in area a-b-e are also acceptable but uneconomical. If other properties (setting time, rigidity, etc.) are essential, similar areas can be determined using the curves of equal setting time, rigidity, etc. Overlapping parts of different areas would determine grouts having simultaneously different properties.

If dry clay is used for the preparation of the grout, the weight of each material is determined from Fig. 6, and the mixing does not present any problem. If clay is kept in a heavy suspension preparation of a given grout "C" follows the successive steps as illustrated by line A-B-C where A is the heavy mix (solids-water = .4); B the mix diluted to solids-water = .2; and C the required grout obtained by adding the cement.

Clay-Sand-Cement Grouts

Attempts to fill large voids by sand grouts have been unsuccessful because of the instability of such mechanical suspension. Sand-cement grouts were found to be more stable, though, when mixed in a standard type grout mixtures, there was always some segregation during the pumping. Use of "activated" or "colloidal" cement grouts, or use of fly-ash plus a dispersing agent has greatly improved the quality of sand-cement mixtures. However, these methods are patented and therefore not readily available. When high strength is not of prime importance, clay-sand-cement grouts are preferred to ordinary sand-cement grouts because of their good stability, low cost, and better adherence to rock.

Grout Properties

Each of the three materials of the grout has a very definite role which governs its proportions in the mixture. Clay is the supporting material. It gives the grout the necessary stability to maintain both cement and sand grains in suspension. High proportion of clay improves the stability but increases the viscosity. The percentage of the clay is therefore kept usually to a minimum. Cement gives the strength to mortar and its setting prevents the ultimate segregation of the grout. It is the most expensive of the three materials and its optimum proportion will therefore vary for each job according to the technical and cost requirements. Sand is the filler and its proportion in the mixture should be as high as possible.

Each of the grouts studied up to now can be prepared with different proportions depending on the grouting conditions. Voids filled by sand-cement-clay grouts are usually of such size that they do not impose any restrictions as to the density or viscosity of the grouts. The restrictions are usually imposed by the grouting equipment and field setup. They have, therefore, a permanent character and the grout is prepared with specific properties and proportions which are kept for the entire job.

Viscosity and stability (and also rigidity and thixotropy) of the clay suspensions used for the preparation of the clay-sand-cement grout can be improved through action of electrolytes or other agents. In the grout itself their action is extremely limited and the viscosity and stability of the grout are therefore directly proportional to its clay content. Because the limitation of the viscosity is the most imperative requirement, grouts have been accepted with a low stability. This can be alleviated on the job by having agitators placed between the mixer and the grout holes. Strength required may vary greatly from one job to another, but the cement proportions must always be sufficient to obtain the hardening of the grout.

Study and Preparation of Clay-Sand-Cement Grout

A trial and error method or a systematic study similar to the one for the clay-cement grouts would be a rather lengthy and expensive process for the search of an optimum grout made of the three different materials. Usually this is not necessary. Experience shows that comparatively few tests are required to obtain if not the best, at least a very satisfactory mixture. In the first applications of the grout, the tendency was to use a very fine sand. Subsequent studies have shown that medium to coarse sand can be successfully suspended and that the resulting grouts have lower viscosity and higher

TABLE #1

Grout	Clay	PROPORTIONS IN WEIGHT Cement	Sand	Water	RATIO $\frac{\text{Solids}}{\text{Water}}$	$\frac{\text{Cement}}{\text{Sand}}$
A	1	2.5	4	10	.75	.625
B	1	3.5	6	14	.75	.585
C	1	3	2	6	1	1.5
D	1	8.2	4.1	9	1.48	2
E	1	10	3	7	2	3.3
F	1	.5	2	2	1.75	.250

strength for the same solids-water ratio. Table #1 gives the proportions of such typical grouts.

Grouts A and B are average strength and economy grouts. Grout C was prepared with a silty clay of low stability which was the only clay available at this particular job. Grouts D and E are relatively high strength grouts. Grout F had no water gain. From Table #1 it can be seen that high strength and high stability grouts have a high solids-water ratio, and a high cement-sand ratio. On the other hand, economy grouts have both low solids-water and cement-sand ratios (usually smaller than 1).

Preparation of a sand-cement-clay grout follows the steps of the preparation of a clay-cement grout. Sand is added last.

Field Procedures

Rock and soil foundation grouting procedures have much in common but also a few differences of characteristics. Among these are the following:

The grouting of rock foundations requires uncased holes except when badly weathered rock is encountered. The average spacing of grout holes is relatively wide (25 feet for primary holes); the maximum pressure is relatively high; and each hole is grouted to refusal. On the other hand the grouting of soil foundations always requires cased holes or perforated pipes; the average spacing of grout holes is closer; the pressure used is lower; and usually only a predetermined quantity of grout is pumped into each hole.

Field procedures for grouting with grouts containing clay do not differ from standard grouting procedures, except for the method of preparation of the grouts.

In the dry method, the clay is dried first, finely ground, and then stored until used. Temperatures as low as possible should be used to dry the clay and, if weather and time allow, the clay can be sun-dried. Various types of mills can be used for grinding the clay: ball, roll or stone mills. As already stated, the inconvenience of this method is in the difficulty of separating the clay grains which have adhered to form small clay aggregates. Therefore, the clay as used in the suspension can be of a much coarser grain size than in its natural condition. On the other hand, handling and measuring of the clay proportion is simplified. If a clay product factory is located nearby, it may

be a ready source of supply and the clay may be delivered in bags; usually the clay is excavated from a nearby quarry.

In the wet method, the clay as obtained from the quarry is stored in water tanks where it is diluted to form a suspension of a very heavy consistency. In these tanks, coarser material contained in the clay settles to the bottom and is thereby eliminated from the clay mixture. The inconvenience of this method is that it requires large storage tanks. Besides, determination of the clay proportion in grouts is more complicated as the density of the stored clay mixture must be constantly checked before being diluted to the required proportions. The advantage of this method is that a much better dispersion of the clay is ultimately obtained as there is no intermediate drying and the final clay suspension is of better quality.

Mixing equipment is the same as that used for the cement grouts. On account of the clay's tendency to lump when poured in water, higher mixing speed is desirable particularly for clay-sand-cement grouts. As already pointed out, the order in which the different materials are mixed is of extreme importance; the correct sequence in the mixing may be the difference between a successful or an unsuccessful grouting mixture. Mixing of the clay always requires special attention. If the dry mixing method is used, care must be exercised to pour the clay slowly while agitating the water.

Pumping procedures with grouts containing clay do not differ from pumping procedures for other mixes. In fact, these grouts are less abrasive than cement or sand-cement mixes. This is especially true in the case of clay-sand-cement mixtures of low water ratio. These grouts have a tooth-paste-like flow which reduces notably the wearing of the pumps and grout pipes.

CONCLUSIONS

A knowledge of the basic properties of clays and clay suspensions, as briefly reviewed in this paper, is indispensable for the study of the grouts containing clay.

The fact that most of the grouts used in pressure grouting are suspension systems leads to the idea that they must be similar in their general properties and methods of preparation and testing.

This is illustrated in the fore study of the clay, clay-chemical, clay-cement, and clay-sand-cement grouts. The methods of selection, design, preparation and testing are succintly described for each type of grout.

It is believed that the information presented is not generally available in the United States. The present knowledge of the grouts and methods of grouting seems to be essentially practical and somewhat limited. It should be further developed both in its theoretical and practical aspects.

ACKNOWLEDGMENT

It was in France, from 1948 to 1951, while with "Sondages, Injections, Forages, Entreprise P. Bachy" and later with "Laboratoires du Batiment et des Travaux Publics," that the writer acquired practical experience and started his work on pressure grouting.

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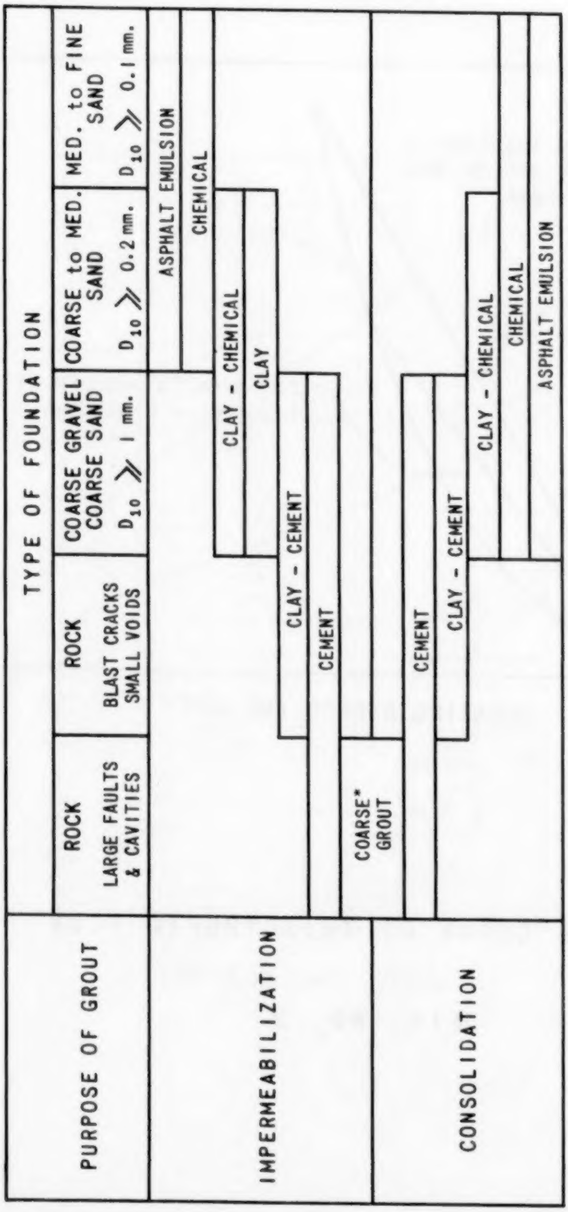
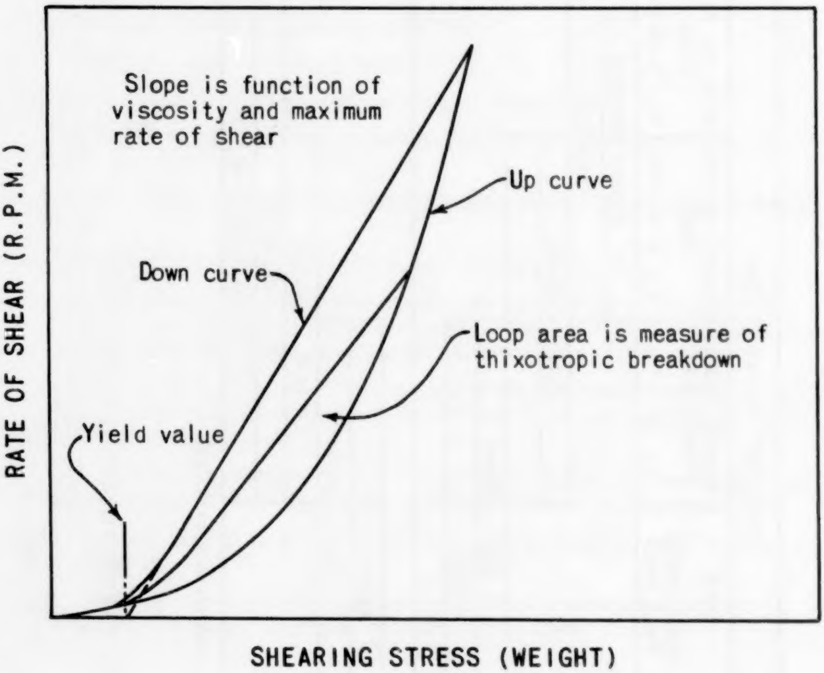


* saw-dust, sand-cement-clay, etc.

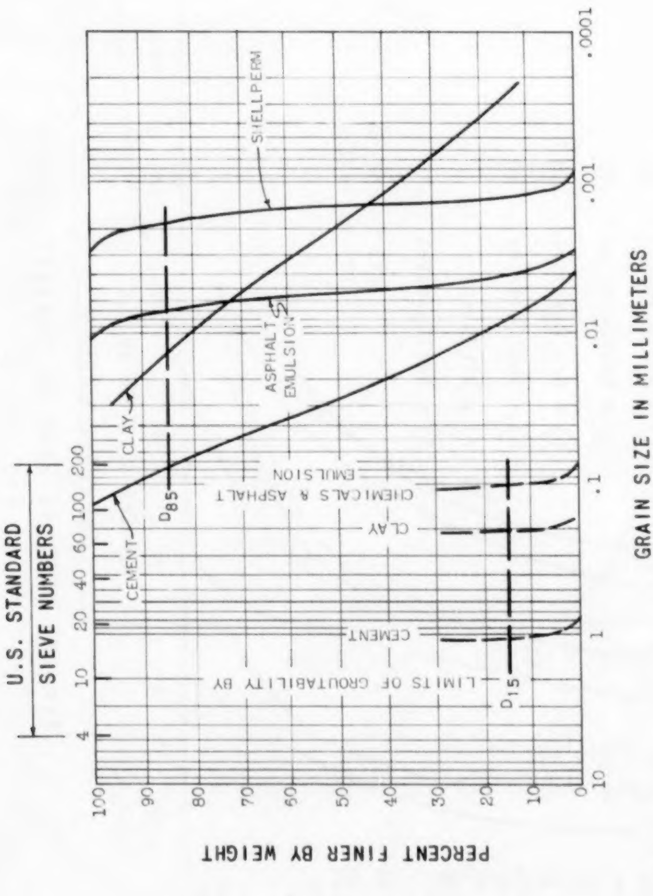
EXTENT OF PRESSURE GROUTS USE

FIG. NO. I



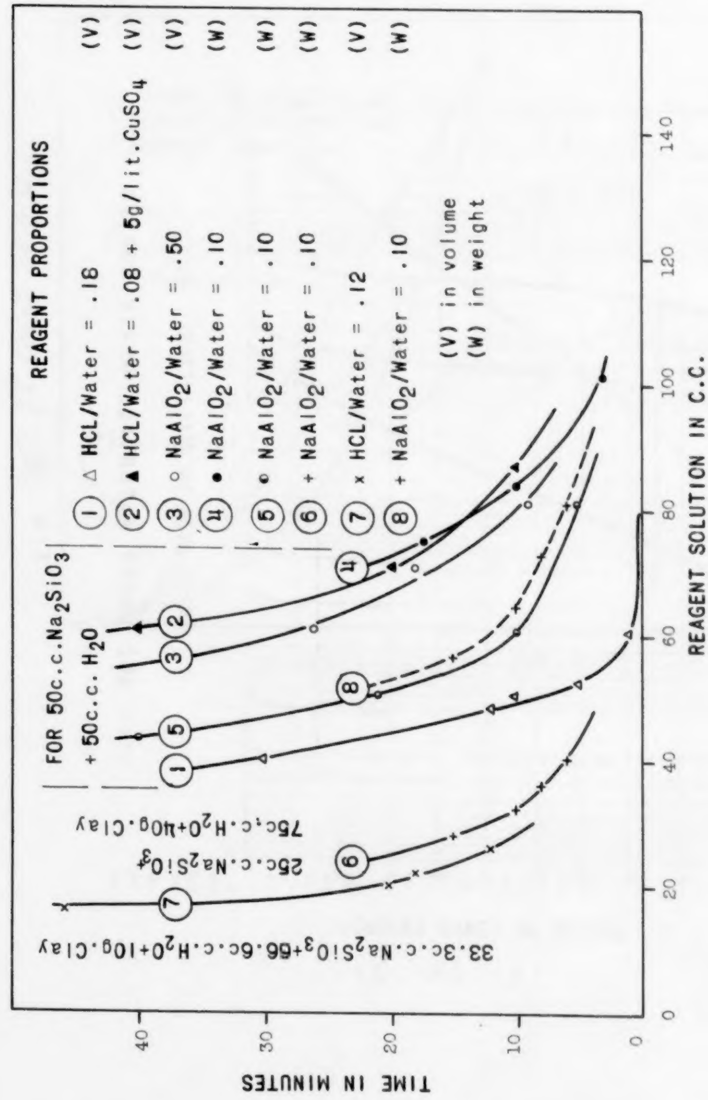
TYPICAL CURVE OF THIXOTROPIC FLOW

FIG. NO. 2



SOIL AND GROUT MATERIALS GRAIN CURVES

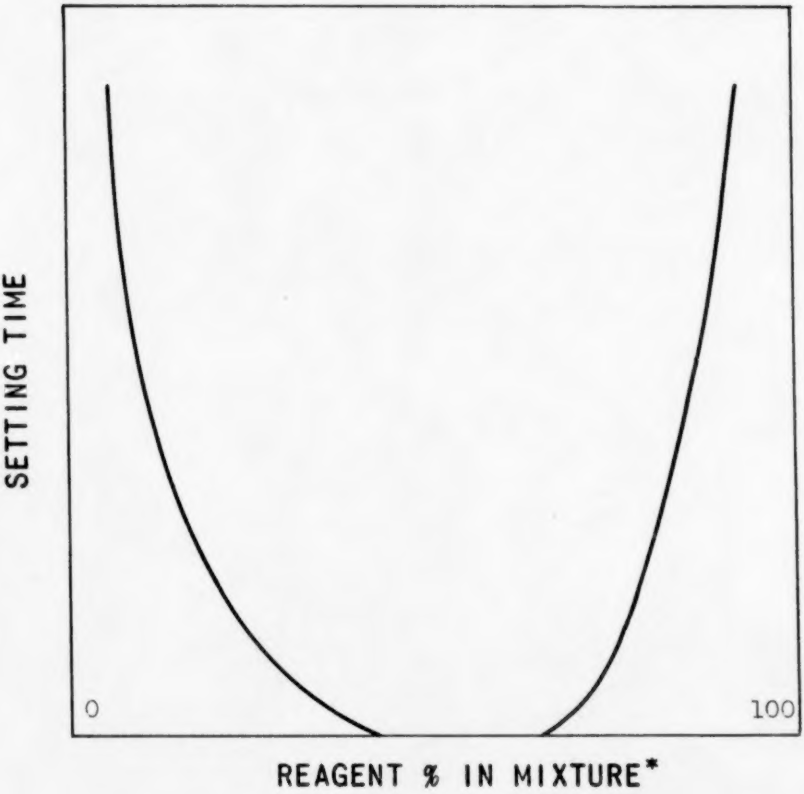
FIG. NO. 3



CHEMICAL GROUTS

TYPICAL SETTING TIME VS REAGENT CURVES

FIG. NO. 4

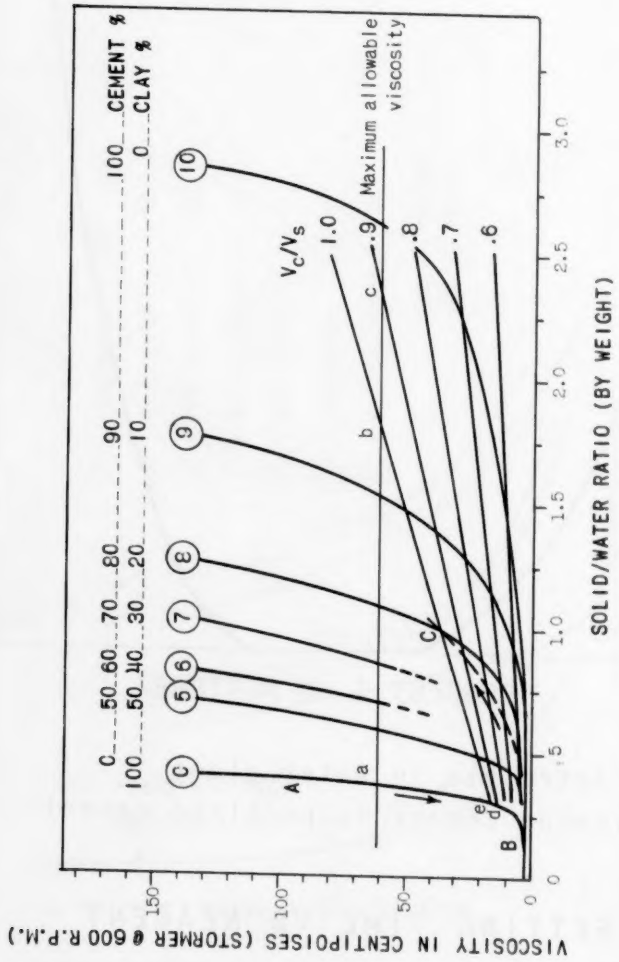


*electrolyte in water glass
alumina cement in portland cement

SETTING TIME VS REAGENT

GENERAL CURVE

FIG. 5



CLAY-CEMENT GROUTS - BASIC CURVES

FIG. NO. 6

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CEMENT AND CLAY GROUTING OF FOUNDATIONS: THE USE OF ADMIXTURES IN CEMENT GROUTS

Alexander Klein,[1] M. ASCE and Milos Polivka,[2] A.M. ASCE
(Proc. Paper 1547)

FOREWORD[a]

The work of the Committee on Grouting of the Soil Mechanics and Foundations Division is divided among four task committees, concerned with (1) bituminous, (2) chemical, (3) cement and (4) clay grouting.

The Task Committee on Chemical Grouting has completed its task and its report was published in the Division Journal, November 1957.

"The Symposium on Cement and Clay Grouting of Foundations" is a "joint venture" of the Task Committee on Cement Grouting and of the Task Committee on Clay Grouting. It includes the following papers:

Proc. Paper 1544 "Cement and Clay Grouting of Foundations: Present Status of Pressure Grouting Foundations" by A. Warren Simonds

Proc. Paper 1545 "Cement and Clay Grouting of Foundations: Grouting with Clay-Cement Grouts by Stanley J. Johnson

Proc. Paper 1546 "Cement and Clay Grouting of Foundations: The Use of Clay in Pressure Grouting" by Glebe A. Kravetz

Proc. Paper 1547 "Cement and Clay Grouting of Foundations: The Use of Admixtures in Cement Grouts" by Alexander Klein and Milos Polivka

Proc. Paper 1548 "Cement and Clay Grouting of Foundations: Suggested Specifications for Pressure Grouting" by Judson P. Elston

Proc. Paper 1549 "Cement and Clay Grouting of Foundations: Pressure Grouting with Packers" by Fred H. Lippold

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Note: Discussion open until July 1, 1958. A postponement of this closing date can be obtained by writing to the ASCE Manager of Technical Publications. Paper 1547 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SM 1, February, 1958.

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a. By Raymond E. Davis, Chairman, Committee on Grouting.

Proc. Paper 1550 "Cement and Clay Grouting of Foundations: French Grouting Practice" by Armand Mayer

Proc. Paper 1551 "Cement and Clay Grouting of Foundations: Practice of the Corps of Engineers" by Edward P. Burwell, Jr.

Proc. Paper 1552 "Cement and Clay Grouting of Foundations: Experience of TVA with Clay-Cement and Related Grouts" by George K. Leonard and Leland M. Grant

These papers are based largely on the results of successful grouting operations for a wide variety of foundation conditions but also are based on the results of laboratory investigations and to some extent on the ideas and opinions of designing engineers who are concerned with foundation problems.

The information contained in the papers has been assembled for the particular benefit of those responsible for the foundation treatment of structures. Recognizing that these papers by no means represent the "last word" in good practice and that there may be many others concerned with foundations who could contribute to our knowledge of cement and clay grouting, discussion is earnestly sought.

SYNOPSIS

The theoretical and practical aspects of the use of admixtures are discussed with respect to grout mixtures of various compositions. Types of admixtures which are useful in grouts, with a brief discussion of the mechanisms involved, are described, as are also test equipment and methods of test for evaluating the contribution of admixtures to grout quality. Typical relationships among grout properties are illustrated.

INTRODUCTION

The usefulness of admixtures in plastic mortars and concretes has been recognized for many years. An admixture consists of any addition to a portland-cement mixture other than cement, water, and aggregate. Utilization of admixtures is rapidly increasing in all fields where cementitious mixtures are employed.

Investigations of the use of admixtures in grouts in progress at the University of California have demonstrated that the desirable properties of grouts may be enhanced through the use of suitable admixtures. The grouts studied in these investigations, as compared to grouts generally employed in foundation grouting, were all relatively thick grouts. Some of the findings of these investigations are presented in this paper, as are also descriptions of special equipment and test procedures which were developed in order to evaluate the contribution of admixtures to grout quality.

Cementitious grouts are liquid mixtures containing portland cement, water, and sometimes sand and admixtures. Foundation grouting consists in injecting such mixtures, usually under pressure, into cracks, voids, and cavities present in such earth formations as rock and gravel for the purpose of sealing

off sources of moving water, or to increase the bearing capacity. Other examples where pressure grouting is employed include contraction joints in dams, railroad ballast, oil wells and construction where the forms are filled with coarse aggregate which is then grouted to produce concrete.

Grout Mixtures

The ingredients are so proportioned as to yield a fresh or hardened grout of the desired properties.

For fine seams in rock foundations, the grout mixtures may be thin, containing only water and portland cement. For large cavities or for consolidating aggregate, the grout mixtures should be relatively thick, containing water, portland cement, and sand. The use of thick grout containing sand contributes to economy, lowered heat generation, improved water retentivity, and cohesiveness. Where sand is employed in a grout mix, however, consideration must be given to problems of segregation and sedimentation, particularly if the grout lacks cohesiveness or if flow velocities are very low.

The scope of this paper is limited to those liquid-grout mixtures which have viscous or Newtonian flow, in which the internal friction is a constant and is represented by a viscosity coefficient.(1) This coefficient is generally expressed as a shearing force. Viscous materials will flow under their own weight. On the other hand, plastic materials require a finite force to initiate flow, but once that force (the yield value) has been exceeded such materials will have viscous flow. While it is understood that during the mixing or pumping there may be encountered a variety of rheological phenomena such as thixotropy, inverse plasticity dilatancy, and work hardening, such phenomena for the most part affect only the mechanical aspects of placement. These phenomena, which have little bearing on the properties of the hardened grout, can be modified through the use of admixtures.

Admixtures are of the following principal categories: (1) Relatively water-insoluble finely divided granular materials which may be inert (rock dust and clays) or chemically active (pozzolans, slags, and natural cement) which contribute to the cementitious properties of the hardened grout, (2) water-soluble compounds (organic or inorganic salts), (3) protective colloids, and (4) gas-producing agents. The water-soluble compounds are generally referred to as surface-active agents, and are usually employed in relatively small quantities.

In this paper, pozzolans and surface-active agents are both considered with respect to their use and to their effects upon the properties of fresh, hardening, and hardened grouts; the methods of evaluating their contribution to grout quality are also described.

Important Properties of Grouts

It was considered that the more important properties of the fresh grouts were (1) consistency, (2) fluidity, (3) water retentivity, and (4) resistance to segregation.

Consistency

The consistency of a grout mixture is related to the viscosity coefficient. The apparatus utilized by the authors for measurement of consistency of grout

and for evaluating the viscosity coefficient or resistance to shear is a consistency meter of the torque type.

Measurement of Consistency

Apparatus

The apparatus for determining consistency of freshly mixed grout, designated as a consistency meter, has the shape and dimensions shown in Fig. 1. The apparatus consists of a turntable platform rotated by a 1/10 HP ratio motor at a constant speed of 60 rpm. This platform lowers to permit the insertion or removal of a 7-in. diameter grout pan. The grout pan serves as the container for the grout which is to be tested. A small key is attached to the bottom of the pan and fits into a recess in the turntable to prevent slippage of the pan during the test. The torque assembly, which indicates the consistency, comprises a spider-type paddle attached to a 23-lb. torsion pendulum with scale attachment graduated from 0 to 360°. The paddle assembly is suspended by piano wire which is rigidly fixed at its upper end and which is of such length as to produce a consistency factor (angular displacement) of 105° when using Baker's grade AA castor oil at 70°F.

Test Procedure

The grout is placed in the pan of the consistency apparatus to a depth of 1 3/4 in., as determined by a gage line machined on the inner surface of the pan. Within the next 1/2 minute the pan is placed on the turntable, rotation is started, and the turntable is raised sufficiently to immerse the paddle in the grout at a position 7/8 in. (clear) above the bottom of the grout pan. At 2 minutes after sample is taken, the angle of rotation or "consistency factor" is noted. The turntable is then stopped, the pan is removed, and the apparatus is cleaned. The stiffer a grout mix, the greater will be its consistency factor.

Fluidity

The fluidity, or essentially the pumpability, of the grout is measured by the resistance to flow through a pipe, channel, or orifice. A convenient method for evaluating fluidity is by the measurement of the time of efflux of a fixed quantity of grout, under falling head through a standardized orifice. For this purpose a flow-cone apparatus was utilized.

Measurement of Fluidity

Apparatus

The apparatus for determining fluidity of grout consists of a flow cone of the shape and dimensions shown in Fig. 2, and a stop watch with a least reading not more than 0.2 second. The flow cone is provided with a point gage, so adjusted that when the discharge orifice is stoppered the total volume to the point gage is 1725 ml.

Test Procedure

The flow cone should be mounted firmly in such manner that the apparatus will be level and free from vibration. The discharge orifice of the flow cone is stoppered, usually by placing the finger over the end of the discharge tube, and the cone is filled with water. One minute before the grout for test is to be placed in the flow cone, the water is allowed to drain from the cone. The cone

is then filled with grout by first sealing the end of the discharge tube and then pouring the grout to the level of the gage point. Immediately thereafter the grout is allowed to discharge and the stop watch is started simultaneously. The watch is stopped at the first break in continuous flow of grout from the tube. The time-interval, in seconds, required for the discharge of 1725 ml. of grout is the measure of fluidity and is designated as "time of efflux."

Water Retentivity

Water retentivity of a grout is indicated by its ability to retain water against vacuum filtration. This property was determined by observing the time required to extract a predetermined amount of water from a fixed volume of grout subjected to vacuum. The water retentivity is considered to be a measure of the cohesiveness of the grout and to be indicative of its resistance to dilution in under-water work and to screening effects of small openings. It has been found that water retentivity, as measured by this method, is closely related to the bleeding characteristics of the grout.

Measurement of Water Retentivity

Apparatus

The apparatus for determining water retentivity of grout consists of a 500-ml Buchner funnel, an 11-cm No. 42 Whatman filter paper, a 250-ml graduated cylinder cut down to contain a volume of 130 ml, a vacuum gage calibrated in inches of mercury to 31 inches vacuum, a vacuum system capable of drawing a constant vacuum of 28 in. of mercury when evacuating a system having a volume of 1 liter, a stop watch having a least reading of 0.2 seconds, and the necessary appurtenances to construct the apparatus shown in Fig. 3.

Test Procedure

Prior to the time of test, a single sheet of 11-cm No. 42 Whatman filter paper is placed on the perforated bed of a Buchner filtering funnel, and the funnel is partly filled with water which is then drawn through the filter paper by means of the vacuum system. When no more water can be drawn out, the stopcock between the filtering funnel and the graduated receiver is closed. The vacuum system is shut off, the receiver is removed, the water is emptied, and the receiver is replaced. The sample of grout for test is poured into the funnel until it is flush with the top. The vacuum system is then started. Within one minute after removal of the sample from the mixture, the stopcock between the filtering funnel and the graduated receiver is open simultaneously with starting of the stop watch. A vacuum of 28 inches of mercury is maintained thereafter. The stop watch is stopped when a specified volume of water has been extracted, and the time indicated on the stop watch is a measure of the water retentivity of the grout mixture; this time is designated as "time of extraction." The usual volume of water removed from grouts with sand is 60 ml, and from grouts without sand is 100 ml.

Resistance to Segregation

Resistance to segregation of a grout can be estimated by measurement of mechanical settling and bleeding. This property is determined by observing the behavior of a fixed amount of fresh grout in a graduated container. Excessive bleeding is considered to be an indication of susceptibility of the grout to an objectionable degree of segregation.

Measurement of Bleeding

This test should be carried out at a room temperature of $70 \pm 5^{\circ}F$. Approximately 900 ml of grout is poured into a 1000-ml graduated cylinder, and the volume is observed and recorded. The amount of bleeding water is determined by observing the water level and the solids level at regular intervals. Depending upon the type of grout tested, the test may be discontinued at either 3 or 4 hours or some longer or shorter period when it is evident that bleeding has practically ceased. After the selected time interval, the supernatant water is poured off from the surface of the grout into a smaller graduated cylinder, where its volume is observed. This "bleeding" is usually expressed as a percentage of the volume of the grout. The solids level indicates the amount of contraction or, if the grout contains an admixture producing expansion of the grout, the amount of expansion.

Method of Laboratory Mixing of Grout

The mixing vessel is placed in position on the stand as shown in Fig. 4. The estimated quantity of water necessary to produce a grout of desired consistency is placed in the mixing vessel, the motor is started, and the rheostat is adjusted so that the speed of stirring is about 500 rpm. The cementing material then is introduced gradually into the mixer over a period of 1 minute. The time of starting this operation is considered as the start of mixing. As the cementing material is fed into the mixing vessel, the speed of stirring is increased gradually until the maximum speed is attained. Three minutes of mixing is sufficient to produce a homogeneous mixture. If the grout is to contain sand, the sand is introduced into the mixture after 3 minutes of mixing, gradually pouring it into the mixing vessel over a period of about 1/2 minute, and continuing the mixing for an additional 2 1/2 minutes. The total elapsed time in this case is 6 minutes from the time of starting.

Admixtures

The trend toward use of admixtures in mortars and concretes, under careful control and supervision, has been significantly accelerated in the last few years. The report of ACI Committee 212 on Admixtures in Concrete[2] has served greatly to stimulate interest in the use of admixtures and to point out the advantages, disadvantages, and considerations involved in the use of admixtures of the several categories in mortars and concretes. Mention is made also of applications of admixtures in grouts. A detailed description of admixtures is given in the ACI Committee report.

Classification of Admixtures

It is not convenient to attempt to classify admixtures on the basis of their contribution to properties of either fresh or hardened grout, inasmuch as a given admixture, such as a water-reducing agent, can have substantial effects upon properties of both the fresh and the hardened grout. It is convenient, however, arbitrarily to classify admixtures on the basis of the quantities commonly used, expressed as a percentage by weight of cementitious material. For the purpose of this paper, a convenient classification consists of:

1. Relatively water-insoluble, finely divided granular materials
    a) Inert
    b) Chemically active

2. Water-soluble compounds
   a) Water-reducing agents
   b) Retarders and accelerators
3. Protective colloids
4. Gas-producing agents

Finely Divided Granular Materials

Inert

A wide variety of finely divided granular materials are available as admixtures to grouts. These are employed in relatively large quantities, up to 50 percent by weight of cementing materials. Among relatively inert materials which make no substantial contribution to cementitious values or strength of hardened grout, are bentonite and other clays, limestone dust, and silica gels. The setting times of grouts are affected by such materials, the rate of setting generally being retarded. The use of bentonite clay or silica gels, which swell upon absorption of water, serves to stiffen the mix without true setting, and can result in various phenomena such as thixotropy. Thixotropy is a characteristic of mixtures which appear thick at slow stirring rates, become thin at high stirring rates, and remain thin for a time after cessation of high-speed stirring. When incorporated in grouts granular materials in finely divided form serve to keep solids in suspension in grouts of high water content and produce grouts of high water retentivity. An attribute of such materials, when sufficiently fine, is their ability to fill cracks too fine to be filled with normal neat-cement grout.

Chemically Active

The chemically active granular materials generally employed in relatively large quantities and in finely divided form include pozzolans, blast furnace slag and natural cement. The pozzolans in themselves possess no cementitious value whereas natural cements and water-quenched blast furnace slags are cementitious in their own right.(3) A pozzolan is a finely divided siliceous material which, when used as an admixture in concrete or grouts, chemically reacts with calcium hydroxide. (a product of portland-cement hydration) to produce cementitious compounds. The pozzolanic reaction proceeds at a slower rate than normal cement hydration, but continues for a much longer period so long as moisture, pozzolan, and calcium hydroxide are present. Pozzolans may be employed in their natural or raw state, or after calcination to a desired "activation" temperature. For materials which are pozzolanic in the raw state, the effect of calcination generally is (1) to reduce the rate of reaction of pozzolan with lime, and (2) to reduce the mixing water requirement of a grout as compared to that of a grout containing the corresponding raw pozzolan. Pumicites, however, are naturally calcined.

An important contribution of some pozzolans, particularly finely ground diatomites, opaline shales, and calcined clays, is reduction in bleeding and segregation. This contribution is enhanced in grouts containing sand. While reduction in both bleeding and segregation is obtained with bentonite clay or silica gel, the potential drying shrinkage of hardened grouts containing these inert materials is far greater than that of grouts containing pozzolans. Furthermore, the reaction products of pozzolanic materials (calcined or otherwise) and lime are insoluble hydrosilicates of lime. These products fill pores and contribute to both water-tightness and strength of the hardened grout.

Also the presence of pozzolans contributes substantially to resistance of the hardened grout to the percolating action of low Ph waters and sulfate waters.

Pozzolanic materials which have been found effective in grout mixtures containing portland cement include:

A - Natural pozzolans
1) Volcanic glasses such as pumicite, pumice, and obsidian
2) Diatomaceous earths and shales
3) Opaline shales and cherts - preferably calcined
4) Clays, such as kaolinite and montmorillinite when calcined

B - Artificial pozzolans
1) Fly ashes
2) Water-quenched boiler slags

For some pozzolans, particularly low-carbon fly ashes, their use in grouts of given consistency may result in water requirements no greater than those for corresponding grouts without pozzolan. However, for grouts containing some pozzolans, such as high carbon fly ashes and volcanic glasses of low fineness, water requirement and tendency toward bleeding may be greater than of grouts containing portland cement alone. Diatomaceous earths or shales also will increase water requirement but will decrease the tendency toward bleeding and segregation. The method of processing diatomaceous earth has a significant effect on its contribution to the properties of grouts. Breaking down the cellular structure of diatomaceous earth by ball-milling improves its properties with respect to water requirement and bleeding.

The amount of pozzolan employed in a grout mixture might be as high as 50 percent, by weight of the cementing material, depending on the nature and fineness of the pozzolan, the richness of the mix and the desired properties of fresh and hardened grout. For a given pozzolan the optimum amount can only be determined by experiment.

Water-soluble Compounds

This group of admixtures are chemical reagents and are employed in relatively minute quantities, from 0.01 to 2 percent by weight of portland cement of, where pozzolans are employed, by weight of total cementitious material. This group includes surface-active agents (water-reducing, air-entraining, and dispersing agents), retarders and accelerators. Such agents may be employed either alone or in combinations to exploit their effects upon specialized properties of fresh and hardened grouts. Many examples of admixtures for various purposes are cited in recent literature.(2)

Water-Reducing Agents

Surface-active agents, when employed in grouts, have a capacity to improve the fluidity in grouts which have a fixed water content. Inasmuch as the quality improves markedly with reduction in water-cement ratio, improved fluidity of grouts contributed by water-reducing agents may be translated into reduction of water content of the mix at a given consistency. While there exist differences in opinion as to the mechanisms whereby surface-active agents (variously termed water-reducing agents, dispersing agents, detergents and the like) improve workability, the fact remains that, in general, the addition of such agents to a mixture at a given consistency results in reduction in required water-cement ratio.

Water-reducing compounds widely utilized in grouts, mortars concretes, are of two basic categories:

1. Calcium, sodium, or ammonium salts of ligno-sulphonates. These ligno-sulphonates are derived from waste sulphite liquor produced as a by-product of the paper-pulp industry. The character of the metallic cation is dependent upon the manufacturing process employed in producing the pulp.

2. Carbohydrates, carbohydrate derivatives, and reaction products of carbohydrate derivatives with various alkylol amines.

When incorporated in cementitious mixtures, these compounds exert side effects other than mere increase in fluidity and workability or reduction in water requirement at given consistency. In some cases these side effects are desirable; in other cases undesirable. For example, many water-reducing agents act as retarders; that is, the rate of stiffening is decreased, and the setting and hardening functions are delayed. In warm weather this retardation may be a desirable feature, but at low temperatures, it may be objectionable.

An additional property claimed for many surface-active agents is that of dispersion, whereby the solid particles in the mixture are separated from each other and are presumed to act as individual particles rather than as clumps of particles, or flocs. It has been found that the effect of many such compounds is to cause objectionable bleeding and segregation. The correction of these conditions may require other admixtures, either for the purpose of acceleration of setting or to compensate in appropriate degree for the tendency to segregate. Nevertheless where these agents are employed in grouts of given cement content and consistency (except for very thin grouts) the unit water content is substantially less with consequent increases in strength, water-tightness, and durability.

Retarders and Accelerators

In grout mixtures, particularly those containing a water-reducing admixture, retardation is rarely required inasmuch as most water-reducing admixtures are in themselves retarders. The simplest expedient for further retarding the setting of a mix containing a water-reducing admixture is to increase the amount of the surface-active agent, or to introduce a finely divided granular material in relatively large amount, if not already present in the mixture.

Accelerators which are employed in cementitious mixtures include calcium chloride, triethanolamine, alkali carbonates and hydroxides, and high-alumina cements. Generally, the commercial suppliers of water-reducing retarders have available products which include accelerators in their composition sufficient in amount to neutralize or at least reduce significantly the effects of the retarding ingredients.

Protective Colloids

Substances such as gelatin, casein, agar, various gums, methylated celluloses, and ammonium stearate are colloids which have the ability to impose upon certain non-colloidal materials some of the properties of colloids. For this reason they are known as "protective colloids." When included in liquid-grout mixtures, they may act to stabilize the suspension and to impart colloidal properties to the smaller solid grains which otherwise would be subjected to gravitational forces only. Hence, when used in appropriate amount,

protective colloids will tend to reduce bleeding and segregation in thin mixes and will to some extent compensate the effects of the thinning of grout mixtures of given water-cement ratio caused by additions of water-reducing and dispersing agents. Other colloids such as bentonite clay, silicic acid, and soaps, although not classed as protective colloids, are effective (through physical or chemical means—particularly swelling) in stabilizing suspensions and reducing the tendency of a grout mix to segregate.

All of the colloidal compounds noted are essentially thickeners, and as such will increase the water requirement of a grout mixture at fixed consistency. In some cases, when used in relatively small amounts, they not only exhibit the effectiveness of protective colloids but also produce an effect similar to that of water-reducing agents. In other cases, the effectiveness of the water-reducing agent is further enhanced by use of a combination of admixtures. This phenomenon is termed "synergystic effect" wherever the combined effect is greater than the sum of the individual effects. The effect is noted particularly in grouts containing finely divided granular materials rich in particles having colloidal characteristics. The effectiveness of the protective colloids, in small amounts, decreases rapidly with increase in sand content. When used in large amounts, the effect of these substances is such that while bleeding and segregation are significantly reduced, the fluidity or the mobility of the mix is impaired and the strength of hardened grout adversely affected.

The use of colloidal admixtures is generally limited to conditions where other methods of overcoming bleeding and segregation are inconvenient or impractical. The use of gas-generating reagents also is effective in the reduction of bleeding and segregation of grouts.

Gas-Producing Agents

Various metals in finely divided form are capable of reacting in alkaline solutions, such as occur in mixtures containing portland cement, in such a manner as to produce hydrogen gas. Some of the metals exhibiting this reaction are zinc, aluminum, calcium, and magnesium. Various chemical compounds such as calcium hydride, hydrogen peroxide, and effervescing salts may also be used for purposes of gas generation. The formation of hydrogen or gas bubbles throughout the mass of the grout results in beneficial effects in the grouting operation and in improvement of grout quality. If unrestrained, free expansion takes place in the grout proportional to the amount of expansive agent. When restrained, the expansive energy in the grout is transformed into pressure. Due to the pressures generated, fine cracks, which otherwise might be bypassed by the flowing grout or be filled with water, may become filled with grout which will set under pressure. The presence of the very small gas bubbles appears to exert a stabilizing influence on the grout mixture in such manner that the grout is cohesive and water-retentive to a degree not exhibited in corresponding grouts made without expansive agents. These gas bubbles cause reduction in bleeding and segregation, regardless of consistency.

Practical Limitations on Use of Admixtures in Grouts

Admixtures of acceptable quality, properly used, may in some cases effect significant economies, and in other cases may make possible construction otherwise difficult or impractical. Though admixtures in general are not a

panacea to the many problems of cement grouting, recent developments indicate that admixtures intelligently used contribute to desirable properties of cement grouts. Inasmuch as admixtures contribute significant effects, and since their effects may vary greatly with the chemical and physical properties of other ingredients of the grout mixture, they should not be used indiscriminately. Rather, careful consideration should be given to the design of grout mixes containing admixtures, and tests to determine their effect should be conducted prior to their use in major construction.

It should be noted that the use of many admixtures in hydraulic cementitious mixtures is covered by patents.

The Structure of Grouts

For theoretical purposes, grouts may be considered as simple two-phase systems. One phase consists of liquid containing all water-soluble constituents and the second phase consists of all water-soluble solids. In mortars and concretes, paste consisting of water and cementitious material is dispersed throughout the solid phase comprising the aggregates. Such a mixture in the freshly-manufactured state is considered to be a plastic material.

Viscous Grouts

In grouts which are pumpable at relatively low pressures, the solids are dispersed more or less uniformly throughout the liquid phase. These grouts act as true liquids and flow under their own weight, without the application of an initial force. That is, the yield value for such mixes is zero, and the flow is said to be viscous, i.e., Newtonian.(1)

Plastic Grouts

Plastic grouts are those which will not flow initially of their own weight but which require an initial force (called the "yield value") to start flow. Although some grouts appear to be viscous with respect to flow, they may consist of two separate grout phases, one viscous and one plastic. In such a case, a relatively small amount of plastic grout having thixotropic properties is distributed throughout the viscous grout. For such a grout the relationship between consistency and fluidity will be a curved function rather than the straight-line function generally occuring in Newtonian flow. This is illustrated in Figure 5 wherein the relationship between consistency and fluidity is shown. With precise measurements, these grouts may show a small shear strength and yield value as indicated by Clark.(4)

Source of Plastic Grout Phase

The sources of change from viscous to plastic flow in thin grouts are first, the presence of fine particles having colloidal properties and, second, the process of precipitation of colloidal gels from hydrating portland cement. Increases in gel concentration result in continuous thickening of a gel and decrease its tendency toward bleeding or segregation. Most experimental studies show that, due to the basic relationship between particle size and surface activity, the fineness of suspended solids affects the viscosity and flow characteristics of their suspensions. For particles greater than 1 micron, the particles are for the practical purposes dispersed, and gravity forces predominate.(1)

One source of colloidal particles (smaller than one micron) is in the unhydrated portland cement. Another source of colloidal particles is in the use of finely divided granular materials or colloidal clay.

A source of colloidal gels is from the hydration of portland cement. Analyses indicate that the various colloidal gels which eventually result in the setting process begin to precipitate to some degree within the first 5 minutes and that their concentration continuously increases. The gels produced by hydration of portland cement are highly stable and when in high concentration require no protective colloids to keep them in suspension. It is reasonable to believe that such gels in themselves act as protective colloids to keep colloidal particles in the grout and even particles substantially greater than 1 micron in stable flocculated suspensions.

Mechanism of Water-Reduction in Grouts

The mechanism of changes in viscosity and fluidity for suspensions, particularly clay and colloid suspensions, is rather well formulated. According to Browning(1) the result of all the forces acting upon suspended particles is an attractive force having its maximum in the particle-size range between 200 and 500 millimicrons (0.2 - 0.5 micron). Because of the high attractive force, particles in this size range form flocculated suspensions which behave in a manner typical of plastic or pseudoplastic flow. For grouts containing a significant amount of colloid, the grout will behave in a manner deviating from Newtonian flow. This condition is an inherent property of the grout and does not explain thickening phenomena occurring in the absence of swelling colloids such as bentonite with increasing age of grout during mixing and agitating.

The structure of a flocculated suspension can be modified through the use of appropriate surface-active agents. Distinctions are made relative to nomenclature of surface-active agents as to whether in a particular case they function as dispersing agents (deflocculating agents), wetting agents (to produce reduced interfacial and surface tension), or protective colloids (to absorb upon particle surfaces and produce a thick hydration layer). The distinctions become vague when it is noted that an agent designated as a dispersing agent significantly increases the fluidity of a simple mixture of water and sand devoid of clay, colloids, or portland cement. It should be borne in mind that in fresh grouts only small amounts of colloid are present.

The activity of surface-active agents in reducing the viscosity and increasing the fluidity of suspensions is well known.(5) This activity is basically a phenomenon related to surface conditions at liquid-solid interfaces. Particles in suspension are presumed to have an electrical double layer about the particle surface, acting to oppose the net effect of all attractive forces between particles. The greater the electrical strength of the double layer, the more stable the state of deflocculation, i.e., dispersion. It is believed that dispersing agents absorb at the particle interface and strengthen the double layer further to oppose attractive forces and thus to insure a more stable state of deflocculation.

The physical significance of the electrical double layer about suspended particles is that the water immediately in contact with the particle is a rigid solid,(6) and such water precludes any shear or movement between the particle and the immediately adjacent molecular film of water. Shear takes place only in the diffuse layer beyond the rigid water layer. In fact then, in a grout

the internal friction and shearing resistance are a function of the diffuse liquid phase. Shear will occur in the diffuse area least affected by interparticle forces and where resistance to shear is a minimum. Correspondingly the mobility (reciprocal of viscosity) of the liquid phase will be a minimum at the solid-liquid interface and will increase in proportion to distance from the interface into the diffuse layer. The ultimate effect of the use of a surface-active agent, for all practical purposes, is an increase in mobility and in fluidity in the most diffuse area of the liquid phase, allowing a reduction in water-cement ratio for consistency equal to that of grout made without the admixture.

Properties of Grouts

As previously stated, the important properties of fresh grouts are consistency, fluidity, water retentivity, and bleeding. The relationships between water-cement ratio, consistency, fluidity, and other properties are shown in Figs. 5 to 9 for grouts containing cement only, cement and pozzolan, and cement, pozzolan and sand. Also shown are the properties of grouts in which a commercial water-reducing agent was employed in the amount of 0.20 percent by weight of cementing material. The portland cement was ASTM Type I, the pozzolan was fly ash, and the sand was a blend of river sands having a fineness modulus of 1.86. The proportions for grouts containing cement and pozzolan were 1 cement to 0.4 pozzolan by weight, and for the cement, pozzolan and sand mixes were 1 cement to 0.4 pozzolan to 1.8 sand. The water-cement ratio is computed as the ratio, by weight, of the net water to the total amount of cementing material.

Consistency and Fluidity

The relationship between consistency and fluidity of grouts of different water-cement ratios is indicated in Fig. 5. This relationship is a curved rather than the straight-line function generally occurring with Newtonian flow. It may be noted that the relationship between consistency and fluidity is practically the same regardless of grout composition, also that water alone falls on the logical extension of the curve. This indicates that the fresh grouts considered here are effectively dilute suspensions of solids.

Effect of Water-Cement Ratio

The effect of water-cement ratio upon consistency and upon fluidity are shown in Figs. 6 and 7 respectively. Also shown in these figures are the data for grouts containing a water-reducing agent.

It is noted that (Fig. 6) a reduction of about 10 percent in water content was obtained for the same consistency through the use of the water-reducing admixture. The use of the fly ash had no significant effect upon the water-cement ratio of grouts for given consistency. On the other hand, the inclusion of sand in the grout mixture caused a substantial increase in water requirement—about 30 percent greater than that for corresponding grout without sand.

In Fig. 7 it is indicated that corresponding benefits are derived through the use of the water-reducing admixture, particularly with respect to effect upon fluidity (or pumpability) at a given water-cement ratio. For a given water-cement ratio the fluidity was increased on the average by about 30 percent.

A comparison of Figs. 6 and 7 indicates, as stated before, that consistency and fluidity are different properties. Consistency, a function of resistance to shearing force, involves a constant for Newtonian flow related to internal friction. Fluidity, a function of rate of flow through a pipe or an orifice, involves a constant related to friction between the grout body and the surface of the pipe. Data shown in these figures indicate that the water-reducing admixture is most effective in reducing the water content of grouts containing sand.

Water Retentivity

In Fig. 8 are shown the relationships between water-cement ratios of grout mixtures and their water retentivity. The lower the water-cement ratio the greater the retention of water. The use of a water-reducing agent improved somewhat the water retentivity of all grouts at a given water-cement ratio, indicating improved stabilization of the deflocculated state of the suspension.

Bleeding

In Fig. 9 are shown the relationships between water-cement ratios and bleeding characteristics of grouts up to the age of 3 hours. The use of the water-reducing agent increased the amount of bleeding even though the water content was reduced. This phenomenon is in accordance with the behavior of a suspension stabilized with respect to the deflocculated state. The greater the degree of dispersion, other things being equal, the smaller the sedimentation volume and the greater the amount of bleeding.

It may be desirable when using a water-reducing admixture to employ in addition an appropriate amount of either a protective colloid or a gas-producing agent, in order to compensate for the increased tendency towards bleeding. Where practical, sand may be used in grout mixtures to decrease bleeding. The effectiveness of the sand in reducing the amount of bleeding is illustrated in Fig. 9.

CONCLUDING STATEMENT

Admixtures may be employed in grouts of various compositions to improve selected properties. The class of admixture to be used will depend upon the desired benefits. Admixtures should be used with caution, and under reasonable control and supervision. Through appropriate though simple apparatus the contribution of admixtures to grout properties is readily determined. The test apparatus and procedures described are applicable for control and design purpose both to laboratory research and to field construction.

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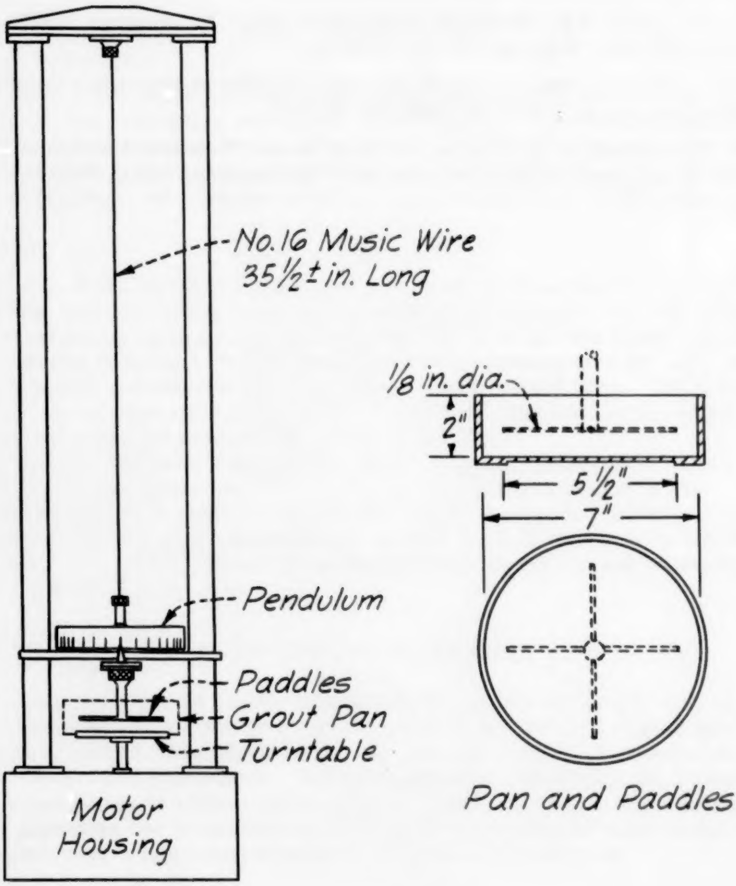

Fig. 1.-- Consistency Meter

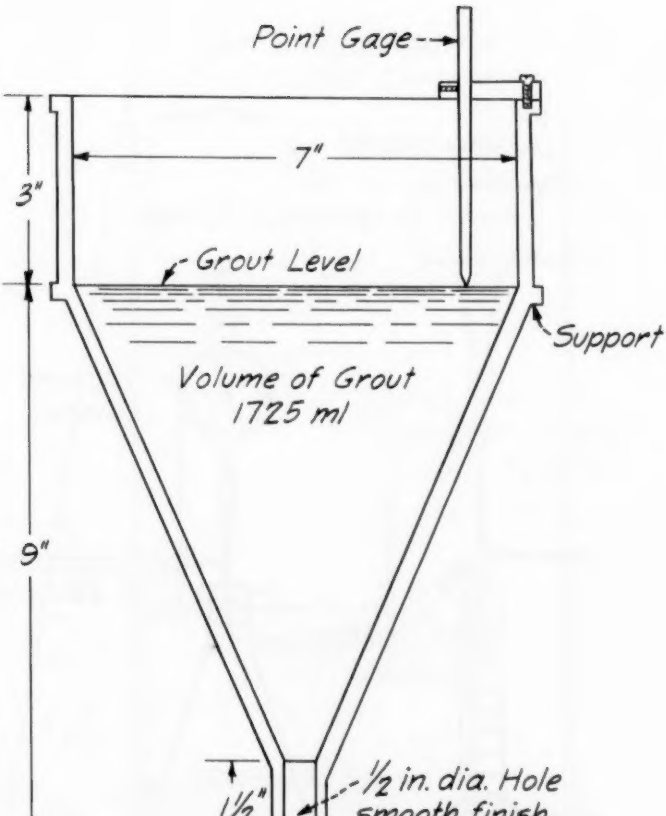

Fig. 2.-- Flow Cone

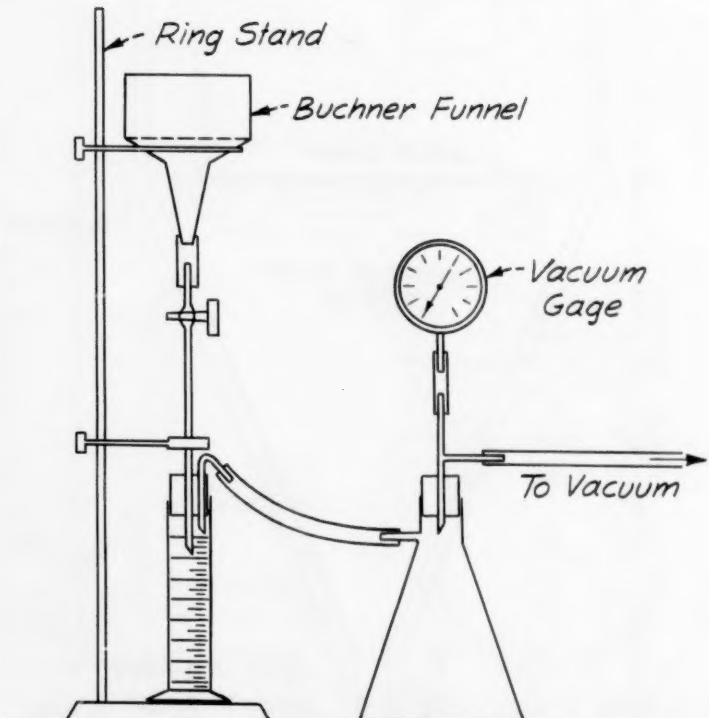

Fig. 3.--Apparatus for Extraction of Water from Grout

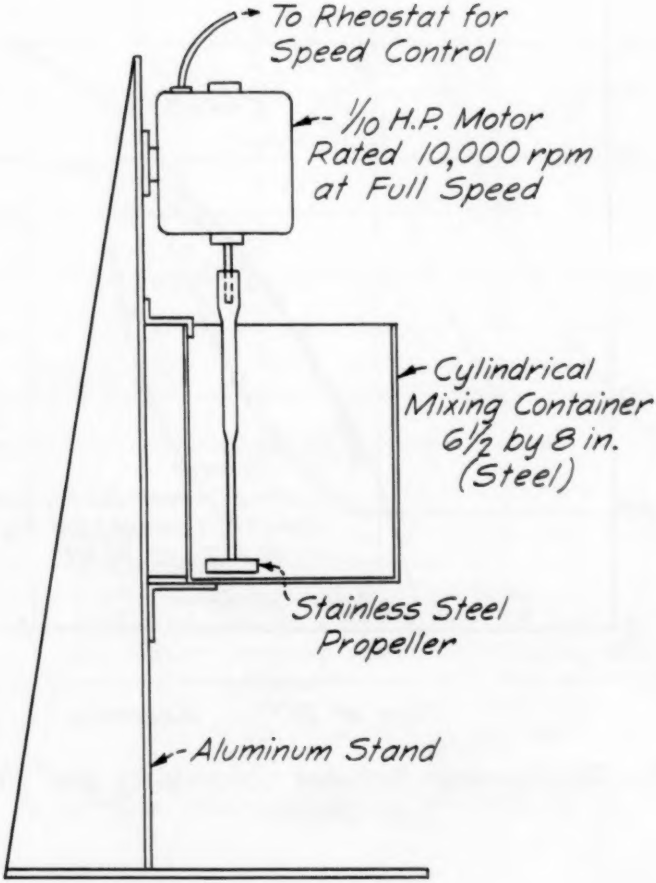

Fig. 4.--Laboratory Grout Mixer

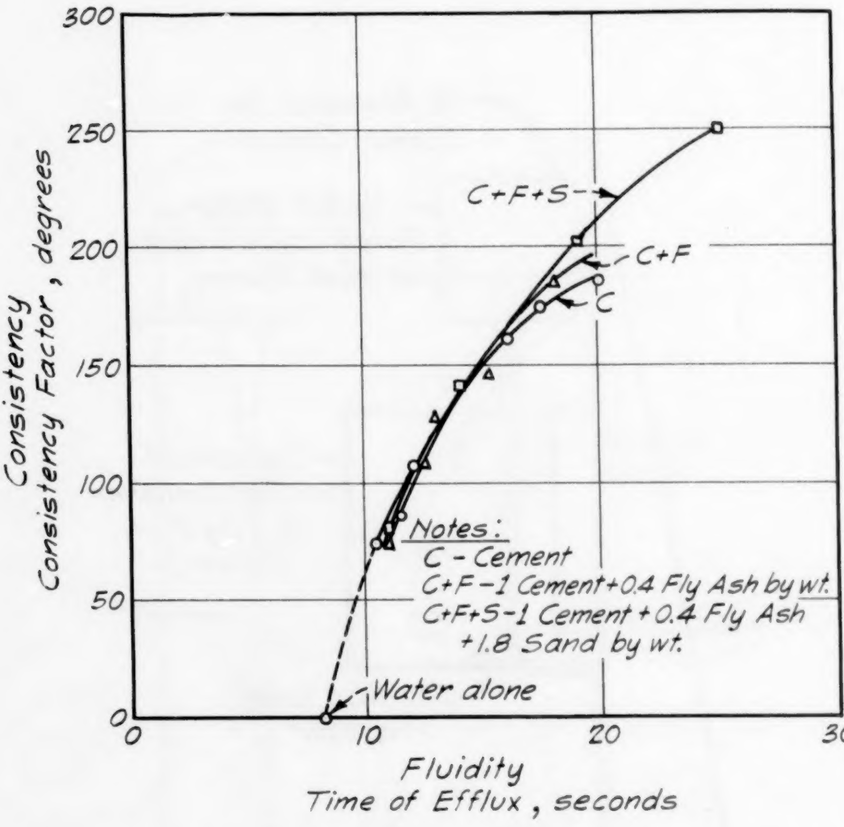

Fig. 5.--Relationship Between Consistency and Fluidity of Grouts

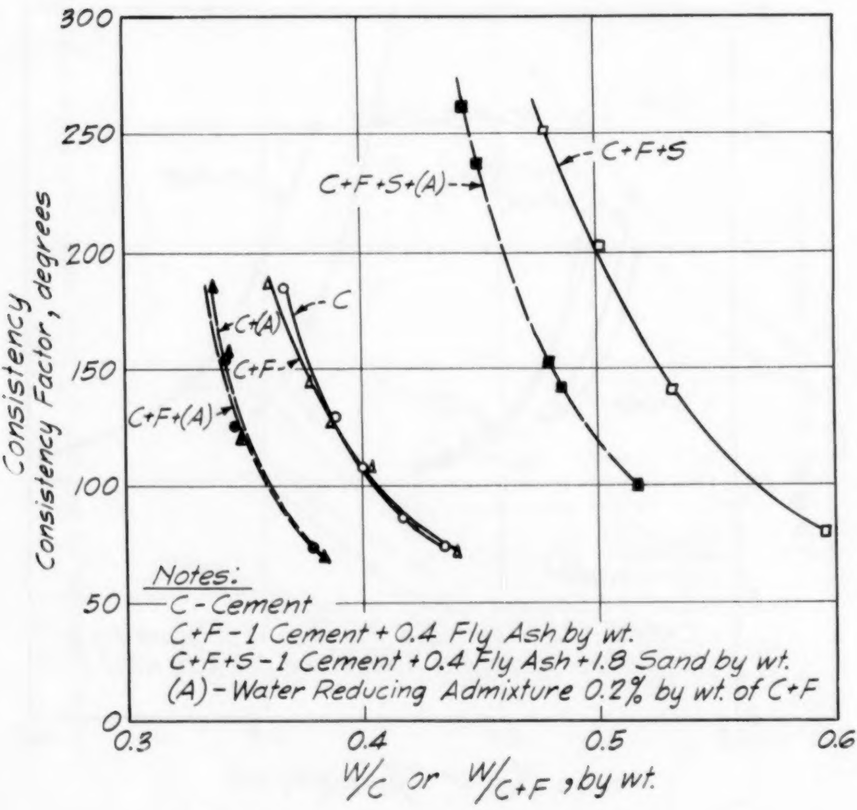

Fig. 6.--Consistency of Grouts

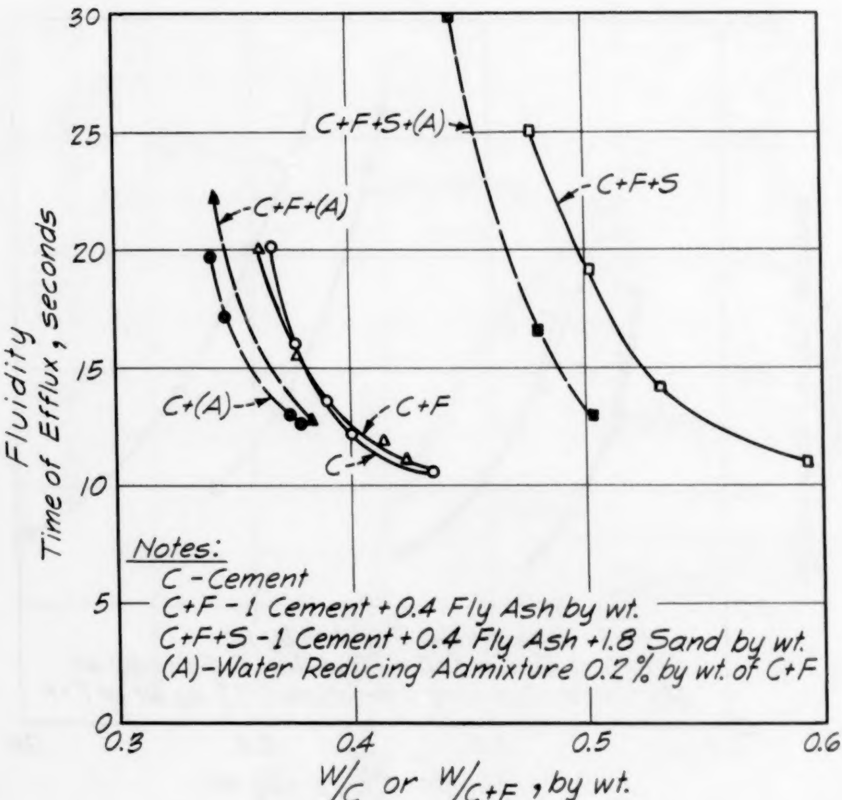

Fig. 7.-- Fluidity of Grouts

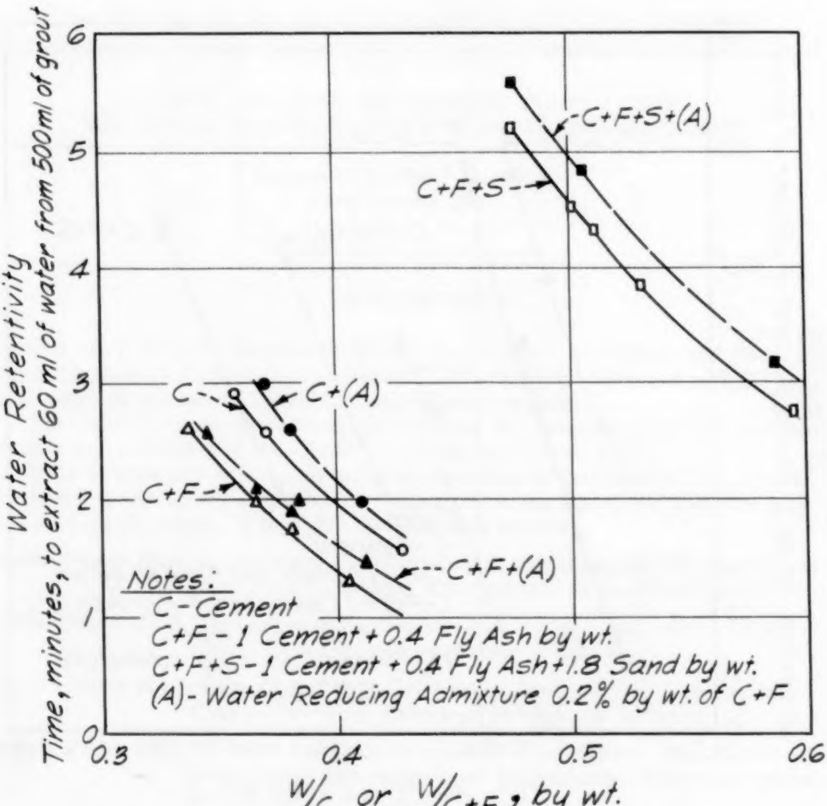

Fig. 8.--Water Retentivity of Grouts

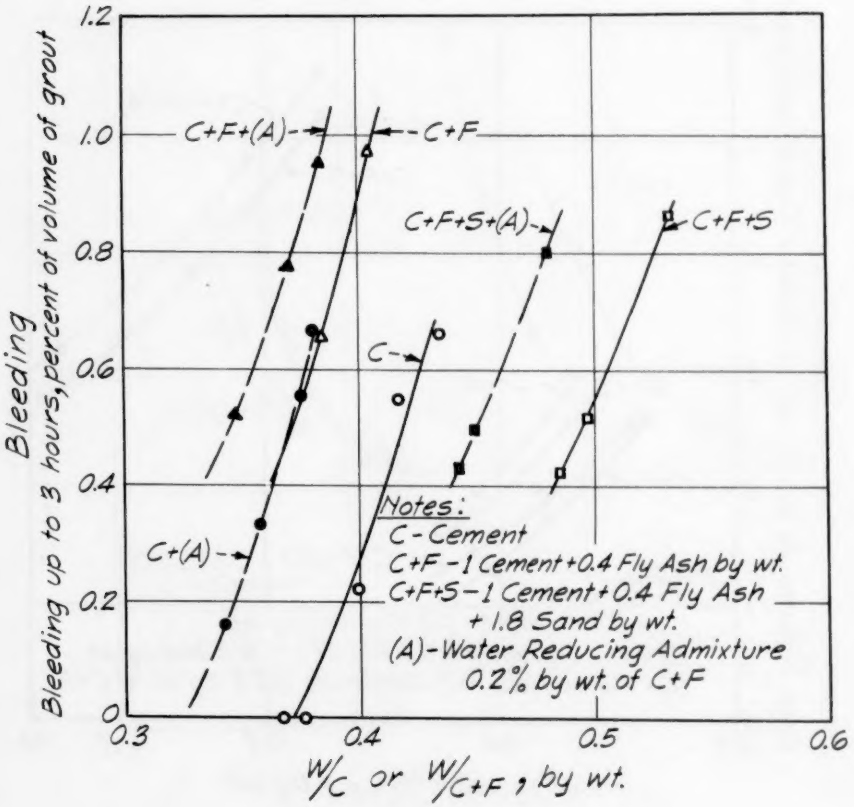

Fig. 9.--Bleeding of Grouts

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

CEMENT AND CLAY GROUTING OF FOUNDATIONS: SUGGESTED SPECIFICATIONS FOR PRESSURE GROUTING[a]

Judson P. Elston,[1] M. ASCE
(Proc. Paper 1548)

FOREWORD[b]

The work of the Committee on Grouting of the Soil Mechanics and Foundations Division is divided among four task committees, concerned with (1) bituminous, (2) chemical, (3) cement and (4) clay grouting.

The Task Committee on Chemical Grouting has completed its task and its report was published in the Division Journal, November 1957.

"The Symposium on Cement and Clay Grouting of Foundations" is a "joint venture" of the Task Committee on Cement Grouting and of the Task Committee on Clay Grouting. It includes the following papers:

Proc. Paper 1544 "Cement and Clay Grouting of Foundations: Present Status of Pressure Grouting Foundations" by A. Warren Simonds

Proc. Paper 1545 "Cement and Clay Grouting of Foundations: Grouting with Clay-Cement Grouts by Stanley J. Johnson

Proc. Paper 1546 "Cement and Clay Grouting of Foundations: The Use of Clay in Pressure Grouting" by Glebe A. Kravetz

Proc. Paper 1547 "Cement and Clay Grouting of Foundations: The Use of Admixtures in Cement Grouts" by Alexander Klein and Milos Polivka

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Note: Discussion open until July 1, 1958. A postponement of this closing date can be obtained by writing to the ASCE Manager of Technical Publications. Paper 1548 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SM 1, February, 1958.

a. This paper is a part of the work of the Task Committee on Cement Grouting of the Soil Mechanics and Foundations Division and is published by permission of the Committee.

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b. By Raymond E. Davis, Chairman, Committee on Grouting.

Proc. Paper 1548 "Cement and Clay Grouting of Foundations: Suggested Specifications for Pressure Grouting" by Judson P. Elston

Proc. Paper 1549 "Cement and Clay Grouting of Foundations: Pressure Grouting with Packers" by Fred H. Lippold

Proc. Paper 1550 "Cement and Clay Grouting of Foundations: French Grouting Practice" by Armand Mayer

Proc. Paper 1551 "Cement and Clay Grouting of Foundations: Practice of the Corps of Engineers" by Edward P. Burwell, Jr.

Proc. Paper 1552 "Cement and Clay Grouting of Foundations: Experience of TVA with Clay-Cement and Related Grouts" by George K. Leonard and Leland M. Grant

These papers are based largely on the results of successful grouting operations for a wide variety of foundation conditions but also are based on the results of laboratory investigations and to some extent on the ideas and opinions of designing engineers who are concerned with foundation problems.

The information contained in the papers has been assembled for the particular benefit of those responsible for the foundation treatment of structures. Recognizing that these papers by no means represent the "last word" in good practice and that there may be many others concerned with foundations who could contribute to our knowledge of cement and clay grouting, discussion is earnestly sought.

SYNOPSIS

Specifications for drilling and grouting work are difficult to write in a positive, clear and lucid style. The great temptation is to adopt one of two extremes, either to attempt to spell out and restrict the work to a rigid and limited set of formulalike conditions or, to introduce and wear out the expression "as directed by the Engineer" for all the work conditions which seem to elude detailed description as to location, quantities, and procedures. An effort is made herein to cover by description and by bid item known or predictable conditions and types of work required. Admittedly, the guide paragraphs are not as descriptive and the minor phases of the work as separated or detailed, especially as to pay items as some contractors might desire, and on the other hand perhaps to an owner they are not restrictive enough or do not appear to insure sufficient control. However, these specifications are the results of and represent experiences of the last 30 years with drilling and grouting work under varying conditions with many different and difficult foundations, and they are the thinking of many of our engineers today on the subject. It is hoped they will be a contribution toward a better working document, equitable and understandable to the contractor, at the same time achieving the design requirements of the Engineer.

INTRODUCTION

Specifications for drilling and grouting work in the past have tended to fall into one of two categories. Either they were so vague and generalized that neither the field engineer nor the contractor could be certain of what was expected or they were so detailed and specific as to smother the intent and compound the confusion. Regardless of the extreme, the result was that the cost of the finished work always became much higher than the estimate and the desired goal was lost sight of. A contractor should be able to easily and readily understand and grasp, without having to treat the specifications as a voluminous encyclopedia, the exact type and quantity of work to be done; the quality required and the conditions under which the work will be carried out; and the factors affecting completion within a stated time limit. The engineer should have a document which defines and establishes his relationship with the contractor and guarantees the completion of the work within the time and to the standards required by the owner and the designer. Grouting deals primarily with the filling of voids in rock or other material and with filling preformed openings in concrete. Weaknesses in foundations are often manifested by irregular shaped openings of indeterminable extent. It is difficult and in most cases impossible to formulate procedures and techniques in advance to cover and anticipate all the problems likely to arise during construction. To establish pressures, water-cement ratios of grout and changes of mixes in advance of the work on each hole is to defeat the very purpose of this work. However, it is still possible to state the estimated quantities, the location of the work, description of the work, and a practical breakdown into separate bid items of related parts of the work. Hidden items, vagueness of description and lack of clarity or frankness on unknown conditions should be avoided. Bid item prices always turn out to be cheaper than extra work orders and negotiations, even when the unknowns are spelled out.

The approach taken in this paper is not new or untried. It has been used to a limited extent by engineers in the United States and Canada. The results to the owners, their engineers, and contractors have been gratifying. However, the temptation of the specification writer is to back away from the ramifications demanded by such an approach. It is simpler to introduce the phrases "as directed," "all or none," "to the extent necessary," or other vague and confusing phrases, to supposedly cover all contingencies and reduce the items in the specifications. It is not the intent of this paper to suggest that the guide specifications contained herein are the solution or the ideal typical specification. There is still room for improvement but it is maintained that the goal to strive for should be a document answering the designer's needs and establishing the work to be done in a clear unquestioned manner which becomes both satisfying and equitable to the owner and his contractor.

Suggested Schedule Items

Item No.	Work or Material	Quantity and Price	
1.	Drill Holes in Concrete	______ lin. ft. per linear foot	@ $________
2.	Drill and Case Holes Through Overburden	______ lin. ft. per linear foot	@ $________

3.	Core Drilling AX Holes, in Rock in Stage Between Depths of 0 feet & 75 feet	_______lin. ft. per linear foot	@ $_________
4.	Core Drilling AX Holes, in Rock in Stage Between Depths of 75 ft. & 150 ft.	_______lin. ft. per linear foot	@ $_________
5.	Core Drilling NX Holes Between Depths of 0 feet and 100 feet	_______lin. ft. per linear foot	@ $_________
6.	Drilling Percussion-Type Holes Between Depths of 0 feet and 35 feet	_______lin. ft. per linear foot	@ $_________
7.	Drilling AX Holes in Rock in Stage Between Depths of 0 feet and 75 feet	_______lin. ft. per linear foot	@ $_________
8.	Drilling AX Holes in Rock in Stage Between Depths of 75 feet and 150 feet	_______lin. ft. per linear foot	@ $_________
9.	Drilling NX Drainage Holes Not More Than 75 feet deep	_______lin. ft. per linear foot	@ $_________
10.	Moving on and off the job with Large Diameter Drills	Lump Sum $_________________	
11.	Drilling 6-inch Diameter Core Holes not more than 50 feet deep	_______lin. ft. per linear foot	@ $_________
12.	Drilling 36-inch Diameter Holes not more than 50 feet deep	_______lin. ft. per linear foot	@ $_________
13.	Metal pipe and Fittings for Grouting and Drainage	_______lbs. per pound	@ $_________
14.	Washing and Pressure Testing	_______hours per hour	@ $_________
15.**	Pressure Grouting	_______sacks *per sack	@ $_________
16.	Connections to Foundation Grout Holes	_______connections per connection	@ $_________

* Some organizations may prefer the words "bag" or "cubic feet" rather than "sacks."

** Mixing and pumping grout mixtures (possible alternate item)-see page b- would include cement, sand, other solids as specified and water. Payment would be made for cubic feet of mixture pumped at direction of Engineer regardless of water-cement or other volumetric ratio of mix. In effect, this would mean that the contractor was being paid for water used in grouting. As a result, the price should be considerably less than normally established in 15.

17.	Thin Wall Tubing for Grouting Contraction Joints	________lbs. @ $____________ per pound
18.	Pressure Grouting Contraction Joints	________sacks @ $____________ *per sack
19.	Hookup to Contraction Joints	________hookups @ $____________ per lift hookup
20.	Excavating 6-foot by 6-foot Shafts in Rock not more Than 50-feet deep	________lin. ft. @ $____________ per linear foot of shaft depth
21.	Furnishing and Installing Uplift Gauge Indicators	________uplift gauge units @ $______________per unit
22.	Furnishing and Handling Cement	________bbls. @ $____________ per bbl.
23.	Portable Telephone System	5 or more plug-in units @ $________per plug-in unit

General

There has been a tendency in many engineering organizations in the past to differentiate in the extent of treatment, size and number of paragraphs, and number of bid items between a specifications for an earth dam and that for a large concrete dam, as well as with other structures requiring grouting work such as tunnels, bridge piers, and building foundations. On a small structure requiring little, if any, grouting and that largely unknown or unanticipated until bedrock is uncovered, the drilling and grouting work write-up would usually consist of a relatively small number of paragraphs condensed as briefly as possible. Where there is question as to the need for the work at all, as to the location, and/or as to the quantities, experience has indicated that lower bid item prices are received and a better working agreement reached with the contractor if a lump sum bid item for moving on and off the job is included. The heading for such work could be called "Requirement for Grouting." On the other hand, a large scale foundation treatment for a structure of major size could be headed up by a paragraph called, "General Plan for Grouting" or "Scope of the Work." For the purpose of this paper, suggested guide paragraphs for drilling and grouting work on a large concrete structure in which most of the conceivable conditions might exist are presented. To fit the specific needs for a smaller structure or a special foundation condition may require tailor making some of the paragraphs by expanding, cutting down, deleting, or adding items and information which will best fit and fulfill the need of the particular job involved. The heading for the section of the general specifications pertaining to foundation treatment can be worded in several different ways to fit the work to be done and the arrangement of other specification sections. Titles could be any one of the following or a variation thereof: "Grouting," "Pressure Grouting," "Drilling, Grouting and Drainage," "Foundation Treatment," etc. Again, titles of the various paragraphs may be varied somewhat to fit specific requirements, however the paragraph headings used herein have been chosen as most representative of the overall work which experience has indicated usually becomes necessary with large structures.

Note: Specific comments concerning the paragraphs are in parenthesis.

FOUNDATION TREATMENT or (PRESSURE GROUTING)

A. SCOPE OF WORK (Name of structure and/or features of a structure, location of each, and type of work to be required. If possible, timing of work in relation to overall construction schedule of other work should be specified.) As an example:

1. Locations and Type of Work - The work includes but is not limited to the following locations or types:
   a. Exploratory work consisting of drilling and sampling of overburden, core drilling of the rock, and drilling of test holes of the (site location)____________________(include site location map - also one may wish to include digging or excavation of test pits or shafts or other specialized exploratory work).
   b. Casing where necessary, drilling, washing and testing as required by the conditions encountered at the site, and grouting the rock under and adjacent to the dam with low pressure shallow grout holes to be followed by final high pressure, deep grout holes.
   c. Drilling and grouting the rock under and adjacent to the embankments of the dikes including the drilling of drainage holes to the extent directed.
   d. Drilling and grouting the rock surrounding the tunnels and shafts.
   e. Grouting the contraction joints and cooling systems (if included) in the dam and__________________(other locations such as spillway, tunnel, power plant, etc.).
   f. Placing mortar or grout by the grouting method to fill all voids between concrete tunnel and shaft lining and rock (if necessary) as required in Paragraph____________(Concrete in Tunnels).
   g. Drilling and grouting in rock around diversion tunnel plug and backfill concrete as required in Paragraph_____________ (Concrete in Tunnels).
   h. Drilling of drain holes.
2. General Program - The program for drilling and for pressure grouting is tentative. The extent of the program will be determined by the conditions developed at the site.

   The low pressure shallow holes for grouting are designated on the drawings as________(letter symbol such as "A," "B" or "C" for each type, location, or purpose) holes, the intermediate holes as ________holes, and the high pressure deep holes as__________holes. Before any concrete is placed in the dam______________and (other structures by name) the near surface rock under the dam and _____ (other structures) shall be grouted by means of "B" holes (description of angles of inclination if required) and to depths of about _____ (varies from 20 feet to as much as 50 feet to fit the particular job conditions, rock, and hydraulic head on structures). The number and spacing of the "B" holes, and the pressures and grout mixes to be used for injections will depend upon the nature of the foundation as disclosed by the excavation, the results of water pressure or

other tests, and the results of the actual grouting operations.

The main cut-off or grout curtain will be completed by high pressure grouting of the deep "A" holes. The drilling and grouting of the "A" holes shall be done (one or more locations - from the foundation gallery in the dam, from the upstream fillet of the dam, and from the cut-off trench for the dikes or earth embankments) as shown on the drawings. It is contemplated that the "A" holes will be drilled at approximately ____________(specify spacing such as: 5-foot) final spacings and to varying depths as shown on the drawings, generally up to a depth of___________(average) feet into rock. If foundation conditions as revealed by the foundation exploration, and the results of grouting operations, indicate that grouting to greater depths is necessary, the contractor will be required to drill some or any number of the holes to maximum depths of ___________ (maximum depth contemplated for exploration and/or grouting—possibly to depths in rock equal to the feet of hydraulic head against the structure) feet and to grout these holes under relatively high pressures. The ____________(spillway and diversion tunnels or other structures by name) shall be grouted with holes approximately 30 feet in depth in rings at 20-foot or less spacings.

The amount of drilling and grouting of all types under any schedule items in this Section that will be required is approximate and the Contractor shall be entitled to no extra compensation above the unit prices bid in the schedule for this specification by reason of increased or decreased quantities of drilling or grouting work required, the time required, the quantities of cleanup, unwatering, or calking required, or by reason of the location, depth, type or nature of the required foundation operations. (Definitions and procedures for exploration, drilling, and grouting are contained in Paragraph D of this section).

B. EQUIPMENT

3. General - Prior to shipment of drilling and grouting equipment to the site, the contractor shall submit drawings and general descriptions of equipment he proposes to use for the approval of the Engineer. (The above requirement might well be waived in the case of a small job where only a general description would suffice; however, it is advantageous for a contractor on drilling and grouting work to satisfy himself prior to bidding as to the equipment and size of crews required for this work. It is recommended that the site of work be visited; core and overburden materials from previous drilling inspected or some experimental drilling done; the local labor and materials market and regulations and housing facilities checked into among other things; all of which information is vital to an intelligent bid).
4. Drilling Equipment - All holes for exploration, grouting, and drainage shall be drilled at the locations, in the direction, and to the depths shown on the drawings or required as grouting operations proceed. Percussion-type drills, rotary-type diamond drills and, if necessary, large size drills, such as a calyx-shot-type drills, will be used in the drilling.

Exploratory holes shall be drilled with standard core-drilling

equipment and rotary drills furnished by the Contractor. Core drilling shall employ bits of the size diameter specified in the schedule items. Plug or non-coring bits may be used at the option of the contractor for grout holes; however, it may be required that up to 10 percent of the schedule item quantities for grout holes and for exploratory holes be core drilled in accordance with the provisions of Paragraph __________(Core Drilling).

5. Grouting Equipment - All equipment used for mixing and injecting grout shall be furnished by the contractor, and shall be maintained in first-class operating condition at all times. The minimum equipment furnished for each grout plant shall include:
   a. Two air-driven, duplex, double acting slush pumps of the long stroke type with large valve openings and removable valve seats and liquid end liners capable of operating at a maximum discharge pressure of__________(state pounds per square inch). Other type equal or better pumping equipment may be used, providing it is specifically approved in writing by the Engineer. (New types of pumping and mixing equipment now on the market are giving good performance and consideration should be given to such equipment at least on a trial basis). The equipment shall be especially designed for grouting service, including the mixing, agitating, pumping and injection of a sanded-cement grout mix if required.
   b. Adequate compressor capacity (if air powered equipment is used—or other capacity if gas, diesel or electric motors are used) to deliver air to each piece of equipment at a minimum pressure of 90 pounds per square inch.
   c. A mechanically operated paddle type, or colloidal (high speed or impeller) type mixer for the mixing of the grouting materials. (The same remark as stated above for new types of pumping equipment applies to colloidal or other type mixers and agitators.)
   d. A mechanically agitated sump or holdover tank with provisions for suitable screens to keep the mixed grout in suspension and remove hardened grout or foreign material not passing a No. 16 U. S. Standard screen. (The arrangement of these facilities to be such that with a gravity-type mixer gravity flow will be provided from the mixer to the agitator. This is extremely important as anyone who has had to clean out a mixer or pump will agree.)
   e. A tank of sufficient capacity and with adequate by-pass pipe and fittings for auxiliary water supply to be used in pressure testing, flushing, and pressure washing operations.
   f. A suitable water meter graduated in cubic feet and tenths of a cubic foot with bypass so water can be measured directly into auxiliary water supply tank. (The cost of gallon measure versus cubic foot measure meters is about the same. Where payment is made by the sack or bag (94 lb. measure) it is much simpler and more convenient to the contractor and to the engineer to use a cubic foot meter.)
   g. Such valves, pressure gauges including gauge savers, pressure hose, pipe, fittings, and small tools as may be necessary to

provide a continuous supply of grout and accurate pressure control. Pressure gauges shall be provided at the grout pump outlet and at the grout hole manifold. (To lengthen the life of the gauges and insure more accurate pressure control, it is desirable to require that gauge savers be provided with each gauge.)

h. Adequate water supply in order that all equipment may be operated at maximum efficiency. (The necessity for an adequate water supply is self-evident to the success of the operation.)

The capacity of the above listed plant shall be not less than 60 gallons per minute (this would naturally vary with the size of the job and anticipated quantities to be pumped) for each pump when operating at a discharge pressure of 100 pounds per square inch. (On small jobs with token schedule quantities, it may be desirable to eliminate the plant capacity requirement.) The inside diameter of the pressure hose and grout supply line shall be not less than 1 1/2 inches and capable of withstanding maximum water or grout pressures to be used. The inside diameter of packer supply pipes shall be not less than 1 inch. (Experience has indicated that 1 inch is the minimum satisfactory operating diameter particularly with heavy grouts at low pressures.)

6. Foundation Displacement Indicators - It may be possible that displacement of the foundation during grouting operations could occur. It will be necessary that such displacement be avoided by use of and careful observation of suitable uplift gauges. The work required is shown on Drawing No.__________. (The type of gauge used, size of hole, pipe and fittings, cementing, the location, and time installed and all other details of this work should be described in specific and sufficient detail to aid the contractor in submitting a reasonable bid.)
7. Communications - The contractor will be required to furnish a portable telephone system approved by the Engineer with a minimum of 10 plug-ins for communication throughout the grouting area. The cost of furnishing communications will be paid for at the unit price per plug-in unit bid in the schedule for "Portable Telephone System."

C. GROUTING MATERIALS

Grout will be composed of a mixture of neat cement and water, with the possible addition of sand, mineral fillers, and fluidifiers or other admixtures. The grout mixes will be varied to meet the characteristics of each hole as determined by conditions encountered.

8. Water - The water used in the grout shall be clean and free from injurious amounts of sewage, oil, acid, alkali, salts, organic matter or any foreign solids, and shall be furnished by the contractor. (If the alternate pay item for grouting "Mixing And Pumping Grout Mixes" indicated on page 5 were used, a clear, understandable method for payment of water would be necessary.)
9. Cement - Cement shall conform to the requirements for Portland cement specified in Section ________(Concrete Work. . . . . - "Cement shall be furnished by the contractor in bulk except that cement

necessary for grouting, finishing, and patching shall be packaged unless directed otherwise in writing by the Engineer." "Portland cement shall conform to Fed. Spec. SS-C-192 a, Type II." On a job where it is known that large quantities will be pumped, it would be a simple matter to set the specifications up so as to have the contractor furnish cement in bulk and provide adequate storage and measuring devices at the grout plant.) To prevent undue aging of sacked cement after delivery, the contractor shall use sacked cement in the chronological order in which it was delivered on the job. Each shipment of sacked cement shall be stored so that it may readily be distinguished from other shipments. The cement shall be free from exposure and lumps due to warehouse set and when used, the empty paper sacks shall be burned. In the event the cement is found to contain lumps or foreign matter of a nature and in amounts which the Engineer considers deleterious to the grouting operations, screening through a U. S. Standard 100 mesh screen will be required. No additional payment will be made for such screening or old cement necessary to be wasted.

10. Mineral Fillers and Fluidifiers - These materials, if required, will be furnished by the Engineer to the contractor at the grout mixer. The cost of using mineral fillers and fluidifiers by the contractor shall be paid for at the unit price bid therefor in the schedule for mineral fillers and fluidifiers. (It is questionable whether this paragraph would ever be needed except for some very specialized condition. If admixtures are definitely contemplated in sizeable quantities, it would be economically advisable to set up a method of payment for furnishing and/or handling such materials in addition to pumping such materials. See page b)

11. Sand - Sand, if required, shall be furnished and handled by the contractor. (The same thought as noted for admixtures should be applied to sand.) The sand shall consist of hard, dense, durable uncoated rock fragments with not more than 5 percent of any deleterious substances. The fineness of the sand required for grouting will be as follows: 100 percent shall pass a standard No. 16 sieve size and 50 percent shall pass a No. 50 sieve size.

11a. Cement and Clay Mixtures - (Cement and clay mixtures have found widespread use on Tennessee Valley Authority foundations composed largely of extremely porous limestone and dolomite. Cement-clay grout is more economical and is satisfactory from the porosity point of view for filling solution channels and caverns in rock subjected to erosion or leaching from hydrostatic pressures. For structural reasons, many authorities do not consider it an adequate sealing or filling material for foundation treatment beneath concrete structures. Cement-clay mixtures have had considerable usage in seepage control of earth embankment foundations and canal banks, and in reservoir rim treatment. Specifications should indicate the source of the clay deposits if in their natural state; supplier if a commercial processed product; processing necessary for the natural material; handling and hauling including quantities with definition and breakdown of the above items into separate bidding columns; equipment necessary for processing and mixing, if different from requirements for neat cement mixes; and separate payment

provisions for furnishing and handling, and for mixing and pumping neat cement mixtures and cement-clay mixtures. Specific requirements, limitations, and procedures are not established here as much of the success of the operation is founded on and fitted to the type of clay used, and its moisture content which often necessitates field research and control. When it is felt that grouting with clays or silts and cements may be required or be beneficial, a specification tailor-made to the available materials and foundation should be developed. It is highly desirable that laboratory tests be run to establish unit weights of clays, moisture content and mechanical analyses of clays, consistencies of mixes, practical field mixes by volume and by weight, initial and final setting times, compressive strengths, permeability and segregation prior to writing the specifications).

D. DEFINITIONS AND PROCEDURES FOR FOUNDATION TREATMENT

12. Location of Grout Holes - Grout holes for shallow or blanket grouting will be laid out in three or more (number established by design requirements) lines of holes with holes on each line offset in relation to the holes on the adjacent lines and spacing between holes initially at 20 feet (adjusted to individual foundation problem). The number of holes for deep or curtain grouting will be determined by progressively reducing the interval between holes through drilling and grouting intermediate holes until the grouting results indicate that a continuous consolidated area or grout curtain of satisfactory tightness has been established.

13. Definitions -
    a. A zone is a series of adjacent rock strata within a predetermined depth of grout curtain.
    b. A section is a horizontal reach along the grout curtain.
    c. A stage is a partial or complete depth of hole for drilling purposes within any given zone.
    d. The "closure" (sometimes called "split-spacing") method is the procedure of locating an additional grout hole midway between two previously drilled and grouted holes.
    e. Packer (often referred to as "stop") grouting consists of drilling a hole to full depth immediately and grouting from the bottom of the hole upward at different depths, settings, or zones by means of a packer set at the required depths.
    f. Stage grouting consists of drilling a hole to a limited depth within a zone, grouting at that depth, cleaning the hole and letting the grout set up, drilling the hole to another limited depth and grouting, and thus continuing in as many stages of drilling and grouting, as may be necessary to secure a satisfactory job of grout within any zone of depth.

    (Paragraph D "Definitions and Procedures for Drilling and Grouting" may well be condensed, or amalgamated in the "Scope of the Work" paragraph. It is thought desirable, however, especially on work involving large quantities and different structures to give as full a description of the character of the work involved as possible. It will be noticed that at times there seems to be minor duplication or repetition, actually

expansion and explanation of Paragraph D within other specific paragraphs in this section.)

14. Exploration Grouting - (Example, presented only as a hypothetical case: During and immediately after dewatering of the site—sites by name and/or location—a series of test exploratory holes shall be put down through the overburden and into the rock for the structures to determine whether the ground water in the overburden materials and/or in the surface foundation rock at or near final grade is under pressure and will flow. If enough water flows from the test holes, the contractor will be directed to proceed with the work of establishing a grout curtain around the perimeter of the excavation area as shown on the drawings. A blanket of natural overburden material not less than 20 feet thick above the porous zones shall be left as a seal until this excavation grout curtain is completed. If the test holes indicate little or no inflow of water, an excavation grout curtain will not be required and the specified earth excavation shall be extended to bedrock surface.

The contractor shall accept the decision of the Engineer in regard to the investigation of leakage and determination of required grouting. The contractor acknowledges in signing the contract documents that no claim based on care of the river or unwatering or maintaining unwatered the site directly or indirectly which may in any way influence the construction schedules of the contractor, will be presented by him for additional payment under the contract. (It is the opinion of the writer that regardless of known or suspected foundation conditions, the problem of and responsibility for "diversion and care of the river" and/or "unwatering the site" should be entirely the contractors' and so established in the specification and contract. Known foundation conditions from exploratory work can be included as information in the specifications as well as the observations of the Engineer. The contractor is then able to bid intelligently and reasonably on unwatering or maintaining the site unwatered from all data and knowledge available to the Engineer.)

15. Shallow Hole Grouting - The shallow hole or consolidation grouting of the foundations will, in general, be conducted in one or more stages of drilling and grouting using the stage method. It may be required that some or any number of the grout holes be grouted using the packer method. The sequence, number and depth of stages of drilling and grouting required to complete any hole in a zone will vary with the conditions encountered.

Should it be determined that a hole be done by the stage method, the contractor shall drill and grout by such method and shall not receive any additional payment above the unit prices quoted for drilling and grouting on account of any number of holes required to be drilled and grouted regardless of whether the packer or stage method is used.

16. Deep Hole or Curtain Grouting - The drilling and grouting of cut-off grout holes shall be done using the "closure" method with packers except where local conditions dictate the use of the stage grouting method. The first holes shall be drilled at 80-foot centers, provided that no drilling or grouting will take place closer than 160 feet until grouting has been completed on at least one hole between any hole to be grouted and any hole to be drilled. Second spacing holes

will be at 40-foot centers, and continued to such spacings as determined necessary to complete the grout curtain. All holes at any one spacing will be drilled and grouting completed before holes at any closer spacing are drilled and grouted. No curtain grouting under concrete structures shall be started until all concrete within ____________(200) feet has been placed to a height of________(50 to 100) feet, unless directed otherwise in writing. (This alleviates the danger of rupturing shallow concrete or horizontal rock layers and permits higher pressures and more complete filling of small seams in the near-surface rock zone. The above spacing procedure may well be modified to fit the particular conditions of the area of foundation treatment. In the case of some soft, cavernous rock, the initial distances between holes might actually be reversed or a ring or hexagonal pattern adopted.)

17. <u>Grouting with Packers</u> - Different grouting pressures will be required for grouting different sections of most of the holes. The grouting shall be performed by attaching a packer to the end of the grout supply pipe, lowering the grout supply pipe into the hole, to the top of the bottom section that is required to be grouted at a different pressure, grouting at the required pressure, allowing the packer to remain in place until there is no back pressure, withdrawing the grout-supply pipe to the top of the next higher section that is required to be grouted at a different pressure, and thus successively grouting the hole in sections at the specified grouting pressures until the entire hole is completely grouted, except that the grouting of the top section shall be performed without the use of a packer. The packers shall consist of expansible tubes or rings of rubber, leather, or other suitable material attached to the ends of the grout supply pipes. The packers shall be designed so that they can be expanded to seal the drill hole at specified elevations and, when expanded, shall be capable of withstanding without leakage, for a period of 5 minutes, water pressure equal to the maximum grout pressures to be used. (The use of packers—stops, plugs, etc.—in grout holes permits the number of separate drill and grout set-ups to be reduced to a minimum, resulting in savings both in time and money involved in any one hole. Some engineers even feel that contractors should bid packer grouting lower than stage grouting.)
18. <u>Regrouting</u> - As the construction work progresses, the development of leakage or the condition of the surrounding foundations may indicate that all or parts of the foundations already grouted require additional grouting. In such event, the equipment shall be returned and additional holes for grouting shall be drilled, and no additional allowance above the contract prices will be made for drilling and grouting such holes or for the expense of moving equipment to other operations and returning to previously grouted areas.

E. <u>DRILLING HOLES</u>

19. <u>Casing and Drilling Holes Through Overburden</u> - Each hole drilled shall be protected from caving, and/or becoming clogged or obstructed. Exploratory and grout holes drilled through overburden shall have a casing keyed into rock sufficiently to exclude all overburden and be watertight. (In some foundation materials, it may be

necessary that the casing be cemented in.) This casing shall be capped and furnished with fittings for water testing or grouting. Any hole that becomes clogged or obstructed for any reason before completion of operations shall be cleaned out in a satisfactory manner or another hole provided by and at the contractor's expense. Casing pipe and other pipe required for exploratory, grout, and drainage holes in overburden and in rock shall have an inside diameter sufficient to accommodate the size of bit required for drilling the particular hole in rock. All metal pipe and fittings required for casing exploratory and grout holes shall be furnished, handled, and installed by the contractor. In casing the exploratory and grout holes in overburden, drive samples at intervals of 5 feet or less throughout the overburden may be required for not to exceed 25 percent of the footage indicated in the schedule for "Casing and Drilling Holes Through Overburden."

20. Drilling Core Holes - The contractor shall perform such core drilling as may be required to determine the condition of the foundation rock prior to grouting or the effectiveness of the grouting operations. Unless otherwise specified, all core holes will later be grouted in accordance with the provisions of Paragraph__________(Description of Foundation Treatment). AX holes shall be not less than 1-7/8 inches in diameter and shall produce cores not less than 1-15/16 inches in diameter. (Larger sizes at higher cost may not be considered essential to an adequate exploration program; however, where a need is justified, they would perhaps be specified as follows:

"NX holes shall be not less than 2-15/16 inches and produce a core of not less than 2-1/16 inches in diameter. The 7-3/4 inch diameter holes shall produce cores not less than 6 inches in diameter."

Maximum depth required for each diameter of hole should be specified. General criteria to establish depth for exploratory holes would be similar to that established for grout holes.

It is the thinking of the writer that exploratory holes larger than AX size are only needed to explore relatively shallow extremely critical foundation conditions. The economical limiting depth for 6-inch diamond drill or 36-inch calyx shot drill holes is about 50 feet.)

All core drilling shall be performed with standard core-drilling equipment using double-tube core barrels capable of producing cores of the diameter specified. Maximum recovery (normally is required to be 95 percent or higher) will be required, utilizing double-tube core barrels of special design equal or better than "M" series for recovery of unpredictable soft or friable materials. (It may be desirable to specify hydraulic-feed on drill machines, or a screw-feed depending on the quality of labor and inspection available and the results desired.) Pressure testing of exploratory and core-drilled holes may be necessary and when required by the Engineer, payment for such work will be made as provided in Paragraph_____ (Washing and Pressure Testing). The drill bit shall be pulled and the core removed as often as may be necessary to secure the maximum possible amount of core. The contractor shall keep, in the manner prescribed by and shall furnish to the Engineer, an accurate

log of all drill holes, including description of all materials of whatever character encountered in the drilling, their location in the holes, and the location of special features such as mud seams, open cracks, soft or broken ground, points where abnormal loss or gain of drill water occurred, and any other items of interest in connection with the purpose for which the core drilling is required. The fact that an inspector may be present and keeping a record of the drilling shall not relieve the contractor from the requirement for keeping an accurage log as described above. The contractor shall furnish wooden core boxes, securely nailed, and for the size of core required including cores up to 6-inch in diameter. The contractor shall place the cores in the boxes in the correct sequence and segregated accurately by labeled wooden blocks according to the measured distances in the holes. No box shall contain cores from more than one hole. Designating marks, hole numbers, and elevations shall be placed on the boxes and along the line of cores. The covers shall be fastened securely to the core boxes, and the boxes shall be delivered at a point designated within________(5) miles of the work.

21. Drilling Percussion-Type Holes - In rock which does not produce mud slurries, percussion-type drilling may be substituted in lieu of rotary drilling, for shallow holes not to exceed 35 feet provided, that prior approval is obtained. (Experience has indicated that small cracks and seams particularly in the igneous and metamorphic rocks tend to plug up from dry hole percussion drilling. The air fed to the cutting edge of the bit drives the rock dust into these fine seams. Therefore, rotary water-fed drilling has been recommended for most foundations.) It shall be necessary that such holes used for grouting are thoroughly washed with air and water immediately before grouting. The washing shall be with alternate jets of air and clean water under continuous pressure, in a manner that will permit free outflow by inserting a smaller pipe and introducing the air and water at the bottom of the hole. (The size of holes required will vary from 1-7/8 inch diameter to 3-inch diameter to fit the particular field condition. The writer feels that small diameter holes should not be drilled when dry-hole drilling is permitted.)

22. Drilling Rotary-Type Holes - The minimum diameter of rotary-type grout holes in rock shall be________inches at the point of maximum penetration. Core recovery will not be required on grout holes and plug or non-coring bits may be used for grout holes. The use of "rod dope," grease, or other lubricants on the drill rods or in the drill holes will not be permitted. (The same reasoning as given for rock dust plugging openings in the rock; i.e., oil or grease is thrown off the drill rods and whipped into small cracks and seams thereby making it difficult or impossible to pump water into the openings.) Holes shall be drilled vertically and, at inclinations from the vertical but not to exceed 30 degrees. (Rock containing high angle or vertical fractures or seams can be more thoroughly sealed off by using angle holes.) Drilling of any hole to a total depth greater than ________feet is not anticipated. Whenever the drill water is lost or large artesian flow encountered on any grout hole, the drilling operations shall be stopped and the hole grouted before drilling

operations are resumed in such hole. Upon completion of drilling of any stage of a hole, it shall be temporarily capped or otherwise protected from entry of foreign material until grouting operations require it to be opened. Redrilling required because of the contractor's failure to clean out a hole before the grout has set shall be performed at the contractor's expense; Provided, that where the grout has been allowed to set by direction of the Engineer, the required redrilling will be paid for at the rate of 50 percent of the unit price per linear foot bid in the schedule for drilling percussion-type holes between depths of 0 foot and 35 feet, regardless of depth. No additional allowance above the unit prices bid in the schedule for drilling exploratory core holes, grout holes, or drainage holes will be made on account of the requirement for interrupting the drilling of holes to permit testing, washing or grouting, or on account of the requirement for cleaning out holes before further drilling, except as specifically covered by Paragraph________(Washing and Pressure Testing). The drilling of curtain grout holes shall be done from the foundation inspection gallery, from the upstream fillet of the dam and from other approved concrete surface elevations. (Location or locations should be definitely defined.) Pipe for grout and drainage holes shall be placed into the concrete or rock a minimum of 18 inches when drilling from the upstream fillet or locations other than the foundation inspection gallery. When drilling from the gallery, pipe shall be placed a minimum of 24 inches into the concrete below the gallery floor or to the top of the first pour lift under the gallery. Unless otherwise directed, the first deep curtain holes under the structures________(include if applicable: "and the first rings of grout holes and the grout holes within each ring in the tunnels and shafts") shall be spaced widely and shall be drilled and grouted before intermediate holes are drilled and grouted, and in this manner, the drilling and grouting of all holes ("and rings of holes and the holes within each ring") shall be completed with such final spacing of holes as the grouting results show to be necessary.

23. Drilling Drainage Holes - Drainage holes shall be drilled in the foundations for the ______________(name of structure or structures), and elsewhere as shown on the drawings. The drainage holes shall be drilled at the angles of inclination shown on the drawings from the foundation inspection gallery (also name other locations) through _____inch diameter pipes embedded 24 inches into concrete or rock, or as shown on the drawings. All drainage holes shall be drilled with diamond drills or rotary drills of a similar type, provided that 10 percent of the quantities shown in the schedule for "Drilling drainage holes" will be required to be core drilled to the standards established in Paragraph________(Core Drilling). In general, the elevation of the bottom of the drainage holes shall be higher than the bottom of the adjacent grout holes, and no drainage holes will be required to be drilled to a greater depth than________feet. The depth of each drainage hole shall be as shown on the drawings. Drainage holes shall not be drilled until all adjacent grout holes within a minimum distance of________(160) feet have been drilled and grouted. The minimum diameter of drainage holes shall be 2-15/16 inches in diameter at the point of maximum penetration. If,

after a given area is grouted and drilled for drainage holes, it is found desirable to drill and grout additional grout holes, the contractor may be later required to open previously drilled drain holes by reaming and/or reopening of existing drainage holes. Such drilling will be paid for at the rate of 50 percent of the unit price per linear foot bid in the schedule for "Drilling Drainage Holes." New or additional drainage holes required by the Engineer will be paid under the applicable unit prices bid in the schedule.

24. Drilling Large Diameter Exploratory Holes - It is contemplated that exploration of rock or concrete requiring the drilling of large size holes may be necessary. The location and the quantities of this work are uncertain, however 6-inch and 36-inch diameter holes may be required. Because of the unknown factors involved, a lump sum item for moving on and off the job (mobilizing and demobilizing) is provided. It may be required that the drilling of 6-inch rotary-type diamond core holes will be required at the same time as 36-inch rotary calyx shot-type holes. The cost of supplying the equipment to the job regardless of the number of holes of different size diameters required at the same time, or the location of work, will be included in the lump sum item. This lump sum item will be paid regardless of whether the actual work is required or not; provided, the contractor is directed to proceed with this work. Drilling requirements of large diameter exploratory holes will be under the applicable provisions of Paragraph_______(Core Drilling). Pumping to remove inflows of water up to a maximum of 500 gpm from the 36-inch holes may be required before and after the hole is completed. Ladders and hoist-operated bo's'n-chairs and illumination for access to the 36-inch diameter holes shall be provided. The contractor shall deliver the cores from all exploratory and core holes to a point designated within________(5) miles of the work site.

F. PIPE FOR FOUNDATION GROUTING

Standard black metal pipe for grout connections may be set in the concrete or foundation rock, except as otherwise provided in Paragraph_____ (_________________________), or the contractor may elect the option of using an expansion type plug for such connections. Pipes for grouting shall also be set over springs, crevices in the rock, faults, or other foundation defects, wherever directed. Grout pipes set in concrete shall end not less than 3 inches inside the finished surface of the concrete. A standard coupling and wrapped nipple to facilitate removal after grouting shall be attached to the grout pipe and shall extend beyond the finished surface of the concrete, as shown on the drawings. The holes left after removal of wrapped nipples shall be filled immediately and completely with drypack in accordance with the provisions of Paragraph_______(Repair of Concrete).

The size of the grout pipe and the depth of holes for setting pipe for foundation grouting shall be as shown on the drawings. The spaces between grout pipes and the rock or concrete into which they are inserted shall be carefully sealed with oakum, grout, or other suitable material to prevent entry of concrete or other materials prior to grouting. All oakum or other suitable material required for sealing shall be furnished by the

contractor. All pipe and fittings to be embedded in concrete shall be cleaned thoroughly of all dirt, grease, grout, and mortar immediately before being embedded in the concrete. The pipe and fittings shall be carefully assembled and placed and shall be held firmly in position and protected from damage while the concrete is being deposited.

All pipe and fittings required for the work described in this paragraph shall be furnished by the contractor. The pipe shall be type 1, class A, standard black pipe in accordance with Federal Specifications WW-P-406. The pipe fittings shall be malleable iron, type 1, in accordance with Federal Specifications WW-P-521b. The pipe shall be cut, threaded, fabricated as required, and placed by the contractor. All pipe and fittings to be embedded in concrete shall have all fittings screwed tight and the pipe braced firmly in position and protected from injury while concrete is being placed.

G. WASHING AND PRESSURE TESTING

25. Exploratory Holes - It will be required that during the drilling of exploratory holes in rock, after drilling is completed, or during the grouting program, the holes be completely pressure tested. When abnormal gain or loss of drill water is observed; caving of the hole or binding of the bit occurs during drilling; or falling of the bit and drill rods as through an open cavity; it may be required that drilling be discontinued and the hole pressure tested either using packers or not. It may be necessary that upon completion of any hole that it be pressure-tested using special double-seal packers set at intervals of 10 feet or less starting at the bottom and continuing to the top of the hole. The equipment, procedures, and any other work necessary will be according to the requirements of the other applicable paragraphs of this Section.

26. Grout Holes - Immediately before the pressure grouting of each stage of any hole is begun, it may be required that the hole be thoroughly washed under pressure and pressure tested. In the event that a washing program is required, at least the nearest two holes in advance of each such hole shall be completely drilled for the same stage and the adjacent holes completely washed to facilitate washing and flushing out of any intervening clay-filled seams, fractures, or solution channels. All intersected rock seams and crevices containing clay or other washable materials shall be washed with water and air under pressure to remove as many of these materials as possible. If practicable, such materials shall be ejected from one or more holes by introducing water under pressure into an adjacent hole. In no case shall such pressure exceed the maximum grouting pressure. All grout holes shall be tested with clean water under continuous pressure up to the required grouting pressure.

Routine water testing of grout holes required prior to grouting will not exceed (20) minutes and the cost thereof shall be included in the unit price bid in the schedule for "Pressure Grouting."

All holes sufficiently tight to build up the maximum required pressure shall be washed at such pressure and the washing shall continue as long as there is any increase in the rate at which water is taken, such increase indicating that fractures and openings are

being reached by the washing operation. Holes in which the required pressure cannot be reached shall be washed as long as there is any increase in the rate of flow or drop in pressure when the pump is delivering a capacity flow. Open holes in which no pressure can be built up shall be washed for such period of time as fracture-filling is being removed, as determined by the escape of muddy water through surface openings or other grout holes.

H. DESCRIPTION OF FOUNDATION TREATMENT

Grouting mixes, pressures, pumping rates, and the sequence in which holes are drilled and grouted will be determined in the field and be under the supervision of the Engineer or his authorized representative.

No curtain grouting under the structures shall be done until all concrete within ______________ feet has been placed to a height of _______ feet.

In no case shall the deepening of any hole preparatory to grouting be commenced before a minimum period of __________ (16) hours has elapsed since completion of the previous stage-grouting in that hole. No second stage grouting shall be conducted within a radius of approximately 40 feet of any hole in which a previous stage of grouting has been completed until the grout in such previous stage hole has set for a period of __________ (24) hours. Grouting at subsequent stages shall conform to the same requirements as to minimum time and distance. (Limitations stated herein become involved in actual procedures and job techniques. To fit the job conditions for a specific structure it may be desirable to redefine, modify, or reduce these limitations.)

The water-cement ratio by volume will be varied to meet the characteristics of each hole as revealed by the grouting operation. In general, if pressure tests indicate a relatively tight hole, grouting shall be started with a thin mix. If an open hole condition exists, as determined by loss of drill water or inability to build up pressure during washing operations, then grouting shall be started with a thicker mix and, with the grout pump operating as nearly as practicable at constant speed at all times, the ratio will be decreased, if necessary, until the required pressure has been reached. Pressures as high as practicable but which, as determined by trial, are safe against rock or concrete displacement, shall be used in the grouting. If necessary to relieve premature stoppage, periodic applications of water under pressure shall be made. Under no conditions shall the pressure or rate of pumping be increased suddenly as either may produce a water-hammer effect which may promote stoppage. The grouting of any hole in which a w/c mixture of less than 3:1 is being used shall not be considered complete until the hole refuses to take grout at the maximum pressure required for that stage of that hole. The grouting of any hole in which a w/c mixture of 3:1 or higher is being used shall not be considered complete until the hole or grout connection takes grout at the rate of less than 1 cubic foot of the grout mixture in 20 minutes if pressures of 50 psi or less are being used, in 15 minutes if pressures between 50 psi and 100 psi are being used, in 10 minutes if pressures between 100 psi and 200 psi are being used, and in 5 minutes if pressures in excess of 200 psi are being used. Should grout leaks develop, the contractor shall calk such leaks, the cost thereof being included in the contract price for "Pressure Grouting."

If, due to size and continuity of fractures, it is found impossible to

reach the required pressure after pumping a reasonable volume of grout at the minimum workable water-cement ratio, the speed of pumping shall be reduced. Following such reduction in pumping speed, if the desired result is not obtained, grouting in the hole shall be discontinued when directed. In such event, the grout shall be allowed to attain initial set, the hole shall be cleaned and after final set of the grout additional drilling and grouting shall then be done in this hole or in the adjacent area as directed, until the desired resistance is built up. If, during the grouting of any hole, grout is found to flow from adjacent grout holes or foundation grout connections in sufficient quantity to interfere seriously with the grouting operation or to cause appreciable loss of grout, such connections may be capped temporarily. Where such capping is not essential, ungrouted holes shall be left open to facilitate the escape of air and water as the grout is forced into the holes. Before the grout has set, the grout pump shall be connected to adjacent capped holes and to other holes from which grout flow was observed, and grouting of all holes shall be completed at the pressures specified for grouting. After the grouting of any stage of a hole is finished, the pressure shall be maintained by means of a stop-cock or other suitable device until the grout has set to the extent that it will be retained in the hole.

The arrangement of the grouting equipment shall be such as to provide a continuous circulation of grout throughout the system and to permit accurate pressure control by operation of a valve on the grout return line, regardless of how small the grout take may be. On open holes, it may be necessary at times to use a single line system to the grout hole. Pressure gauges and adequate valves will be required at the pump and at each hole to insure control, bypass, and shutoff as required by the inspectors. The equipment and lines shall be prevented from becoming fouled by the constant circulation of grout and by the periodic flushing out of the system with water. Flushing shall be done with the grout intake valve closed, the water supply valve open, and the pump running at full speed.

Grouting pressures to be used in the work will vary with conditions encountered in the respective holes. As a safeguard against rock or concrete displacement or while grout leaks are being calked, reduction of pumping pressures or the discontinuance of pumping may be required. It is anticipated that pressures will range from_______psi to______psi but in no event will pressures in excess of______psi be required.

I. <u>CONTRACTION JOINT GROUTING</u>

27. <u>Tubing for grouting contraction joints</u> - Systems of thin wall steel tubing and fittings and grouting outlets shall be placed in the contraction joints in the dam____________and elsewhere, as shown on the drawing. All tubing, fittings, and grouting outlets required for permanent installation in the contraction joints, all nails, tie wire, asphalt emulsion for sealing purposes, and temporary supports required for the installation of the grouting systems shall be furnished by the contractor. The tubing will be furnished in random lengths and shall be cut to length and formed by the contractor. Before the contractor purchases any tubing, fittings, or grouting outlets he shall furnish the Engineer a list of suppliers and samples of such materials and secure approval of the type of seals, tubing, fittings, and grout outlets he proposes to use.

All tubing and fittings to be embedded in concrete shall be cleaned thoroughly of all dirt, grease, grout, and mortar immediately before being embedded in the concrete. The tubing and fittings shall be carefully assembled and placed and shall be held firmly in position while the concrete is being placed. Great care shall be exercised to insure that the two companion members of each conduit-box-cover grouting outlet are maintained in accurate alignment and position with respect to each other and that each member becomes an integral part of and moves with the concrete mass to which it is anchored. The method of attaching the first member of each grouting outlet to the forms and, in turn, the second member to the first is shown on the drawings. This method shall be adhered to accurately unless it is modified by specific instructions of the Engineer.

Where grout tubing terminates at an exposed concrete surface, the tubing shall be fitted with a wrapped nipple and kept capped until grouted as shown on the drawings. After the grouting operations have been completed, the contractor shall remove the grout nipples in the face of the structure, and all holes left after the removal of the nipples shall be filled immediately and completely with drypack in accordance with the provisions of Paragraph __________ (Concrete).

Care shall also be taken to insure that all parts of the systems are maintained free from dirt and other foreign substances. After each lift of the grouting systems is placed and before any concrete is placed around it and at such other times as may be necessary the tubing shall be tested by forcing a current of air under pressure through it after which it shall be immediately temporarily capped or otherwise closed to avoid the possibility of any foreign substance entering it until it is pressure grouted.

A continuous flow of water shall be circulated in the ungrouted supply headers of the contraction joint systems in the near vicinity of any foundation hole being grouted. Should grout enter a contraction joint system as a result of the foundation grouting, the contractor shall clean out the system before the grout has taken its initial set, by filling the system approximately half full of water through the supply header of the bedrock lift or the lift above, admitting air under pressure through the lowest return available, and opening and closing the air valves in such a manner as to produce waterhammer in the system.

Any tubing that becomes clogged or obstructed before final acceptance of the work, due to any cause, shall, if practicable, be cleaned or opened. For any plugged tubing which the contractor fails to open or to replace, the contractor shall pay to the Engineer, as fixed, agreed, and liquidated damages, the sum of five dollars $_______($) per linear foot of the total length of tubing which is thereby made ineffective.

28. Pressure grouting contraction joints and cooling systems - As soon as possible after the concrete has cooled the desired amount, the contraction joints and the embedded cooling systems in the dam and elsewhere, as shown on the drawings shall be pressure grouted with Portland cement grout. All grout for pressure grouting contraction

joints shall consist of neat cement mixed with water. Cool water shall be used in the grout mixture to prevent quick setting of the grout, and water having temperatures above 70° F shall not be used. The cement to be used for pressure grouting the contraction joints shall be furnished by the contractor. (A specification for special cement for this purpose should be included). All grout shall be pumped unless otherwise permitted. The program of grouting, the time when each lift of a contraction joint shall be grouted, the grout mixture used, the pressure applied, and all other details of the grouting operations shall be in accordance with these specifications and under the supervision of the Engineer or his authorized representative.

The grouting of the contraction joints shall be done singly or in groups, and in separate successive lifts beginning at the foundation and finishing at the top of the dam. The grout feeder pipe shall extend close to the supply pipe for each lift of joint being grouted and shall have a connection with each supply pipe regulated by a valve. When more than one joint is being grouted at the same time, the grout shall be supplied in rotation by batches or in such quantities as necessary to fill each joint lift at approximately the same rate and to complete the filling of all joint lifts at the same time. The grouting of each lift of joint shall be completed before the grout takes its set in the grout pipe system but shall not be grouted so rapidly that the grout will not settle in the joint lift and in no case shall the time consumed in filling any lift of joint be less than two hours.

The contractor shall provide all necessary facilities such as catwalks, ladders, and platforms to enable the inspectors to observe the grouting operations at each joint, and for quick and convenient passage from joint to joint as required. The contractor shall supply all labor, tools, and material required in setting any dial gauges or other devices used to indicate opening or closing of the joints during the grouting operation. The contractor shall provide a telephone communication and signal system for use during the washing, testing, and grouting operations. Before any lift of a joint is grouted, it shall be washed thoroughly with air and water under pressure and shall be filled with water for 24 hours. Immediately prior to being grouted, the water shall be drained from the joint lifts to be grouted.

During the grouting operations, two or more lifts, of adjacent ungrouted joints at the same level, shall be filled with water to the level of the top of the lift being grouted. As the grouting of the lift of the joint nears completion, the lift of the joint immediately above the lift being grouted shall be filled with water. Valves shall be used to control the flow of water into and from each joint. All accessible leaks discovered prior to grouting and all leaks occurring during the grouting operations shall be calked or stopped in a satisfactory manner. Immediately after a grouting operation is completed, the water shall be drained from the joints in the lift above but shall not be drained from the adjacent ungrouted joints at the same level until 6 hours after completion of the grouting operation. Joints, pipes, or formed drains into which grout has leaked shall be

thoroughly washed out by alternately filling them with water and draining them until all grout has been removed, and all headers and outlets for each such joint, pipe, or drain shall be tested and shall be cleaned before the joint, pipe, or drain shall be considered to be thoroughly washed out. The grout shall be pumped into the bottom header of the piping system for the lift. During the grouting of each lift of joint, the outlet end of the pipe at the top of the lift shall have a riser pipe about 5 feet in height and a valve near the top of the riser. The valve shall be left open until the lift is filled and only grout of proper consistency for retention in the joints remains in the riser, whereupon the valve shall be closed and the required pressure applied. The water and thin grout shall be bled from the top of the joint lifts and pressure shall be applied as many times as is determined to be necessary to force all excess water from the grout. After the system ceases to take an appreciable amount of grout, the required final or residual pressure shall be maintained on the grout in the joint lift until the grout has taken its initial set. The simultaneous application of grout at two or more points in any one system shall be made if determined to be necessary.

After the cooling systems have served their purposes, the pipes of the cooling systems embedded in the concrete of the dam shall be filled completely with grout.

29. Hook-up to contraction Joints - Hook-up to vertical contraction joints or joint lifts in the dam and to the joint or joints in the______ ____________________ will be required to: grout the various lifts; to wash out and test the joints prior to grouting and; if necessary to rehook the lifts during or after the grouting operations. Each contraction joint is divided into one or more separate grouting lifts from bedrock to the top of the dam. Each contraction joint grouting lift contains its own seals, supply piping system and venting system. The item of the schedule for hook-up to contraction joint grouting lifts shall include the cost of furnishing all labor, materials, tools, and equipment necessary to provide access to the joint lift headers, to install and remove temporary pipe lines to each grouting lift, to wash and test each lift under pressure, and other incidental work to and during the grouting. Methods and procedures for hooking up to contraction joint grouting lifts and for washing and testing the lifts shall conform to the applicable provisions of paragraph_________, (Pressure grouting contraction joints and cooling system).

J. PROTECTION OF WORK AND CLEANUP

No grouting will be permitted within__________(100) feet of installed drainage or perforated pipe, tile pipe, gravel filters, or other drains. The contractor shall maintain a flow of water through the drains likely to be affected within__________(100) feet of grouting operations in such a manner that the entire drain or drainage system can be thoroughly flushed clean. In case leakage of grout, drill cuttings or other foreign material into drains does occur, the contractor shall remove all such material from the drains affected by washing or by other means and the cost of any such work shall be included in the unit price bid in the schedule for "Pressure Grouting." During drilling and grouting operations, the contractor shall take such precautions as may be necessary to prevent drill cuttings, oil,

wash water, and grout from filling up and plugging gallery gutters, drains, pipes and sumps, or defacing any permanently exposed surfaces, or damaging any part of the permanent structure. No grouting will be done until all blasting within ____________(200) feet of the grouting operation has been completed. The contractor shall clean and remove all waste upon completion of his drilling and grouting operations.

K. MEASUREMENT AND PAYMENT

30. General - The contract prices for the various items of work and materials, as described under this section, shall constitute full compensation for mobilizing and demobilizing and furnishing all equipment, material, labor, engineering services at the job site and incidental work in accordance with these specifications; Provided, that a lump sum item will be paid for moving on and off the job with large diameter exploratory drills as described in Paragraph______ (Drilling Large Diameter Exploratory Holes).
31. Casing and Drilling Holes Through Overburden - Measurement for payment will be made from the collar of the hole at the exposed surface to the actual depth drilled and cased. Payment for "Drilling and Casing Holes in Overburden" will be made at the unit price per linear foot bid in the schedule, which unit price shall include the cost of furnishing all labor, materials, pipe, casing, tools, and equipment required for casing the holes; taking drive samples at intervals of 5 feet or less for a linear footage not to exceed 10 percent of this bid item; removing all materials from the casing and maintaining the holes open and clean until no longer required; providing valves and fittings to the top of casing so that the hole may be used with or without a packer for grouting; and all incidental work connected therewith. The pipe casing may be left in place or removed at the option of the contractor, except that all pipes or holes remaining shall be completely filled with grout at the expense of the contractor. If the pipe casing is removed, it shall remain the property of the contractor.
32. Drilling Core Holes - Core drilling will be measured for payment from the collar of the hole at the exposed surface to the actual depth, up to the depth of holes drilled at the direction of the Engineer. Payment for "Core Drilling" described in Paragraphs________and __________ (   ) will be made at the unit prices per linear foot bid in the schedule for core drilling "AX" or "NX" holes in stage in rock between depths stated in the schedule, which unit prices shall include the cost of furnishing all labor, materials, tools and equipment required for drilling the holes; removing the cores; keeping accurate logs of drill holes; boxing and labeling and transporting the cores; and all incidental work connected therewith.
33. Drilling Holes In Concrete - Grout holes drilled in concrete will be measured for payment from the collar of the hole at the exposed surface to the actual depth of grout holes drilled in the concrete as specified between the surface of the structure rock foundation and the floor of the gallery, or upstream "heel" (fillet) of the structure from which grouting is done; or between the surface of the structure rock foundation and collar of the hole at the top of masonry from which grouting is done. If the Engineer gives permission for the

contractor to place pipe for lining these holes instead of drilling, the length of pipe so installed will be paid for under Item No. 13 of the schedule in lieu of payment for drilling. Payment for "Drilling Grout Holes in Concrete" will be made at the unit price per linear foot bid in the schedule, which unit price shall include the cost of furnishing all labor, materials, tools and equipment required for drilling the holes; drilling the holes through concrete including steel or other embedded materials; core drilling under the applicable provisions of Paragraph__________(Core Drilling) up to 25 percent of the unit quantities provided for this item; maintaining the holes free from obstructions until grouted; and all incidental work connected therewith.

34. Drilling Percussion-type Holes - Measurement for payment will be made from the collar of the hole at the exposed surface of rock or concrete to the actual depth up to the full depth stated in the schedule. Payment for "Percussion-Drilled Grout Holes" will be made at the unit price per linear foot bid in the schedule for drilling percussion-type holes in concrete or rock which unit price shall include the cost of furnishing all labor, materials, tools, and equipment required including washing of holes prior to grouting, and all incidental work connected therewith.

35. Drilling Rotary-type Holes - Payment will be made only for the linear feet of hole drilled for expansion type plug or for pipe nipples used in grout holes, but not for both. No payment will be made for pipe used for pipe nipples unless specifically ordered in writing by the Engineer. (Depending to some extent on equipment used and anticipated quantities of packer grouting contractors may prefer to use pipe nipples grouted or calked into the rock or concrete with which to anchor drilling equipment and grout pipe manifolds, or to use a packer from which to hook on manifolds.) Measurement for payment for determining stage depths of drilling grout holes in rock will be made from the collar of the hole at the exposed surface of the rock or concrete to the actual depth drilled into the rock foundation and concrete as directed. Payment for drilling grout holes in rock will be made at the applicable unit price per linear foot bid in the schedule for "Drilling grout holes in rock in stage" between the depths specified in the schedule, which unit prices shall include the cost of furnishing all labor, materials, tools, and equipment required for drilling the holes; maintaining the holes free from obstruction until grouted; and all incidental work connected therewith.

36. Drilling drainage holes - Holes will be measured for payment from the surface of concrete or rock from which the drilling starts to the full depths of holes actually drilled. Except as otherwise provided for redrilling and drilling to a depth greater than__________feet, payment for the drilling of drainage holes will be made at the unit prices per linear foot bid in the schedule for "Drilling drainage holes in stage in rock," which unit price shall include the cost of all labor, materials, plant, and all operations required in drilling the holes and maintaining them free from obstruction until the work is completed.

37. Drilling large diameter holes - The cost of moving to and from the site of the work with all equipment, parts, and materials including

labor and incidental work required for the drilling of any hole from a nominal 3-inch diameter core drilled hole up to and including 36-inch diameter rotary diamond type and/or calyx shot-drill type core holes is included in the lump sum item bid for this work; provided, it is directed in writing to be done by the Engineer. Large diameter holes in rock or concrete will be measured for payment from the surface of the rock or concrete from which drilling is started to the bottom or full depth of hole. Payment for drilling large diameter holes will be made at the unit prices bid therefore in the schedule for (List specific diameter called for under each item such as: 3-inch, 6-inch, 36-inch, etc.)______________, which unit prices shall include the cost of labor, materials, tools, equipment, pumping, drilling, providing access, and all incidental work connected therewith.

38. Metal pipe and fittings for grouting and drainage - Metal pipe and fittings will be measured for payment by the pound. The contractor shall furnish and install pipe of sufficient diameter for casing of exploratory and grout holes through overburden to permit drilling the required depth of holes in rock, but no separate payment will be made for the pipe and the cost thereof shall be included in the unit price per linear foot bid for "Casing and drilling exploratory and grout holes in overburden," whether the pipe is recovered or not. Payment for furnishing and placing metal pipe and fittings for foundation grouting and drainage will be made at the unit price per pound bid in the schedule for "Furnishing and placing metal pipe and fittings for foundation grouting and drainage," and only for the pipe and fittings permanently installed and left in place as directed, and no additional allowance above the unit price bid in the schedule will be made on account of the varying size, or length, or number of pipes required.

39. Washing and Pressure Testing - Measurement for payment for washing and pressure testing will be determined from the time the pump starts to inject water into the hole until the time the pump is stopped as determined by the Engineer. It is the intent of the specifications that the item for Washing and Pressure Testing will be used only in the event that the contractor is notified in writing that the Engineer decides that soft materials can be washed from two or more holes and a scheduled washing and testing program is necessary.

Routine water testing of holes prior to grouting will not exceed 20 minutes and the cost thereof will not be paid under the item "Washing and Pressure Testing" but shall be included in the unit price bid in the schedule for "Pressure Grouting."

Payment for the work of washing and pressure testing under a scheduled program will be made at the unit price per hour bid in the schedule which work shall include the cost of labor, materials, equipment, water, tools, pipe and fittings, packers, including any number of settings in each hole, right angle or other high pressure net nozzles or orifices, and all incidental work connected therewith. No payment will be made under any other unit prices for the work required for washing and pressure testing. All work shall be done in the presence of the Engineer.

40. Pressure Grouting - Mixing and Pumping Grout Mixtures - (Possible alternate item - Engineers and contractors are looking more favorably on the possibility of making payment for the entire fluid mixture, mixed and pumped, including the water used in such a mixture. Some arguments are that such a payment provision would provide a sounder bidding basis for a contractor particularly when he had to consider possible low grout consumption and thin grouts. For example, take a 10:1 grout or 10 parts of water to 1 part of cement. The Engineer could exercise closer control and more personal treatment of each hole with the contractors' whole hearted cooperation as he would be receiving payment for the work required. There are cases where he may be required to pump large volumes of water and little or no cement.)

Pressure grouting will be measured for payment on the basis of the number of bags or sacks of Portland cement or sand (94 pounds) actually injected into the grout holes.

Payment for pressure grouting will be made at the contract price per sack bid in the schedule for "Pressure Grouting" except for grouting to be paid for as specified below. The price for "Pressure Grouting" shall include the cost of furnishing all materials, labor, tools, and equipment required for the grouting including routine pressure testing of grout holes with water; plugging or calking leaks on water and grout; and use of packers (stops); except that payment for furnishing and handling cement will be made at the unit price per barrel bid for "Cement" in the schedule; and payment for connections to each grout hole will be paid for as described in Paragraph ____________, "Connections to holes." No payment will be made for grout or for cement used in grout lost due to improper anchorage of grout pipe or connections, for grout or cement used in grout lost due to negligence on the part of the contractor, nor for grout or cement used in grout rejected by the Engineer because of improper mixing or failure to calk leaks. In measuring sand for payment for mixing and pumping in grout, one 94 pound bag of sand will be considered as 1 cubic foot dry measure. (An item should be provided in the schedule for furnishing and handling sand for grouting - 100 percent will pass #16 mesh, 50 percent will pass a #50 mesh - by the cubic foot or the cubic yard.)

41. Connections to holes - Payment will be made only once for that section of any hole in which packers are used, regardless of the additional number of times packer settings are made or the same hole is hooked onto while using a packer, and regardless of the volume of water or grout actually injected into the grout hole. Payment for holes which are required to be stage drilled and grouted or in which unusual packer settings are required as determined by the Engineer, will be made once for each time the hole is hooked onto for grouting or pressure testing after completion of the stage depth of drilling.

The number of connections or separate grout holes requiring hook-ups, as shown in the schedule item "Connections to Holes" is only approximate, and the contractor shall be entitled to no additional compensation above the unit price bid in the schedule by reason of the number of connections actually required to complete the grouting operations as specified in Paragraphs.

42. Tubing for Grouting Contraction Joints - Except as otherwise provided, payment for all work described in this paragraph will be made at the unit price per pound bid in the schedule for furnishing, handling and installing metal tubing and fittings for grouting contraction joints, which unit price shall include the cost of furnishing unloading, hauling, storing, handling, and installing the tubing and fittings; of protecting the tubing from injury and clogging; and of removing the nipples in the face of the structure and filling, with dry-pack, the holes left by the removal of the nipples. Payment will be made only for the tubing and fittings, including grouting outlets, actually installed, and the computed weight for payment will not include the weight, of nails, tie wire, asphalt emulsion for sealing purposes, or temporary supports.

43. Pressure grouting contraction joints - Measurement for payment for pressure grouting contraction joints and cooling systems will be made on the basis of the number of sacks of cement actually forced into the joints and systems or required to fill permanent pipes. Payment for pressure grouting contraction joints and cooling systems will be made at the unit price per sack bid therefor in the schedule, which unit price shall include the cost of all labor, materials, plant and operations required for the grouting, except that payment for furnishing and handling special cement for grouting contraction joints will be made at the unit price per barrel bid therefore in the schedule, and payment for hooking onto each contraction joint grouting lift will be made as provided in Paragraph ______________. (It may be desirable to have a separate bid item for cement of special fineness such as "Cement for contraction joint grouting shall be secured from a mill equipped with air separating equipment. The cement shall meet the following requirements for finess: 100 percent shall pass a U. S. Standard 100 mesh screen and 98 percent shall pass a U. S. Standard 200 mesh screen.")

44. Hookup to contraction joints - Payment for hook-up to contraction joint grouting lifts and ______________________________ will be made at the unit price per hook-up bid therefor in the schedule, which unit price shall include the cost of washing and testing each complete contraction joint grouting lift and ______________________________. Payment will be made only once for hooking up to each contraction joint grouting lift and ______________________________ regardless of the number of times the same lift or joint may have to be hooked onto for washing, testing, grouting, or regrouting to insure a complete and satisfactory grouting operation for each contraction joint. The item for hook-up to contraction joint grouting lifts does not include the hooking of cooling coils in the dam and ______________________.

45. Uplift Gauge Indicators - Payment for furnishing and installing uplift gauge indicators will be made at the unit price per uplift gauge unit bid in the schedule, which price for each unit shall include the cost of drilling the holes; furnishing pipe and fittings and other miscellaneous metal work and materials required to construct the gauges and all labor and installation of gauges including anchoring, sealing, grouting, and miscellaneous work required. It may be necessary that the gauges be extended to a gallery or tunnel in the structure. In the case of such extension, the contractor will be paid

for the pipe and fittings and valves required at the applicable unit price per pound bid in the schedule for the wrought iron metal pipe, fittings and valves.

46. Furnishing and Handling Cement - Measurement, for payment, of sacked cement will be on the basis of the number of sacks of cement used at the mixer. Cement for mortar and grout shall be furnished by the contractor. To prevent undue aging of sacked cement after delivery, the contractor shall use sacked cement in the same order in which it was delivered to the jobsite or railhead. Each shipment of sacked cement shall be stored so that it may readily be recognized from other shipments. Bulk cement bins shall be weathertight. The contractor shall empty and clean bins when so directed. The cement shall be free from lumps and shall be otherwise undamaged when used in grout. If the cement is delivered in paper sacks, empty paper sacks shall be burned. (Type of cement, Federal Specification Number, shipment designations, inspection, and acceptance tests should all be written into this paragraph.) Measurement, for payment, of bulk cement will be on the basis of batch weights at the batching plant or mixer. Any cement, either bulk or sacked, used for grouting or other miscellaneous work will be measured for payment in the most practicable manner. One barrel shall be considered as 4 sacks of sacked cement or 376 pounds of bulk cement. Payment for furnishing and handling cement will be made at the unit price per barrel bid therefor in the schedule, which unit price shall include the cost of rail and truck transportation of the cement from the mill to the jobsite, the cost of storing and, if necessary, rehandling the cement, and the cost of batching or handling the cement at the mixer.

47. Portable Telephone System - Payment for furnishing, installing and maintaining a portable telephone system for use during the washing, testing and grouting operations, and for use during the work of washing, testing and grouting of the contraction joints will be made after two plug-ins, consisting of receiver and transmitter, have been installed and are determined to be working satisfactorily. It will be required that a phone unit be located at each grout plant and grout pump and at the header or hole being grouted. Payment will be made on the unit price per plug-in unit bid under the schedule item and only for the plug-in units actually required in service by the Engineer. The price per plug-in unit will include all apparatus, wire, insulators, poles and incidental materials, labor and equipment necessary for clear, undistorted voice communication between distances up to 1 mile and will include the cost of maintenance of the entire system throughout the life of the contract.

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CEMENT AND CLAY GROUTING OF FOUNDATIONS: PRESSURE GROUTING WITH PACKERS

Fred H. Lippold,[1] M. ASCE
(Proc. Paper 1549)

FOREWORD[a]

The work of the Committee on Grouting of the Soil Mechanics and Foundations Division is divided among four task committees, concerned with (1) bituminous, (2) chemical, (3) cement and (4) clay grouting.

The Task Committee on Chemical Grouting has completed its task and its report was published in the Division Journal, November 1957.

"The Symposium on Cement and Clay Grouting of Foundations" is a "joint venture" of the Task Committee on Cement Grouting and of the Task Committee on Clay Grouting. It includes the following papers:

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Proc. Paper 1548 "Cement and Clay Grouting of Foundations: Suggested Specifications for Pressure Grouting" by Judson P. Elston

Proc. Paper 1549 "Cement and Clay Grouting of Foundations: Pressure Grouting with Packers" by Fred H. Lippold

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Proc. Paper 1550 "Cement and Clay Grouting of Foundations: French Grouting Practice" by Armand Mayer

Proc. Paper 1551 "Cement and Clay Grouting of Foundations: Practice of the Corps of Engineers" by Edward P. Burwell, Jr.

Proc. Paper 1552 "Cement and Clay Grouting of Foundations: Experience of TVA with Clay-Cement and Related Grouts" by George K. Leonard and Leland M. Grant

These papers are based largely on the results of successful grouting operations for a wide variety of foundation conditions but also are based on the results of laboratory investigations and to some extent on the ideas and opinions of designing engineers who are concerned with foundation problems.

The information contained in the papers has been assembled for the particular benefit of those responsible for the foundation treatment of structures. Recognizing that these papers by no means represent the "last word" in good practice and that there may be many others concerned with foundations who could contribute to our knowledge of cement and clay grouting, discussion is earnestly sought.

SYNOPSIS

The technique of effectively, yet economically, pressure grouting a foundation varies with the foundation material and the special local conditions. Pressure grouting with packers is one of the methods that is proving effective, especially in weaker foundation material. Packer grouting in strong rock at greater depths is also an improvement over other methods but for somewhat different reasons. This paper will briefly explain the technique and attempt to show the advantages as well as point out the disadvantages of pressure grouting with packers. For the purpose of explaining the procedure it will be assumed that the grouting is being performed to create a water barrier, but with minor modifications the packer method is also effective for consolidation or backfill grouting and can be used for placing a clay grout or a sanded cement slurry.

INTRODUCTION

Pressure grouting a rock foundation to create a water barrier, whether it be under a dam or around a shaft or lock has been developed materially in the last 25 or 30 years. Whether the grout is being placed through a single line of drill holes or through multiple lines,[2] the problem of placing the grout in the water bearing areas is essentially the same. There are several methods for controlling the point of injection, the most common of which was, and still may be, stage drilling and grouting.[2] The "stage" method certainly was a big improvement over the older full depth method of grouting the entire length of a deep hole at one time by simply hooking onto the pipe nipple calked into the collar of the hole. Packer grouting may be as large an improvement over the

2. See Table I.

Table I

Type of grouting	Description
Multiple lines	Two or more parallel lines of grout holes, usually 10 to 20 feet apart, to create a grouted zone of some width
Full depth	Grouting the entire length of hole at one time by connecting to nipple at top
Stage	Drilling and grouting hole in predetermined depth increments until final depth is reached
Closure or split spacing	Drilling and grouting widely spaced holes, usually 40 feet apart, before intermediate holes are drilled and grouted. Procedure is repeated until "closure" (as shown by minor acceptance of grout) is reached.
Zone	Procedure where a predetermined depth of grout curtain is completed before deeper holes are drilled and grouted. This is repeated, usually not over three times, until final depth of curtain is reached.

stage grouting method. Conditions may be encountered occasionally when a combination of the two methods can be used advantageously.

A brief review of pressure grouting methods used and developed in the past 25 or 30 years may be in order here. The greatest single factor affecting pressure grouting procedures was the adoption of the positive displacement pump to inject the grout slurry. Although that was a vast improvement over the old pneumatic placing method, it also created some new problems. How to avoid damaging the foundation and still use the higher pressure available with a displacement pump became a serious problem. Modern designs and the ability to treat a foundation make it feasible to build hydraulic structures on broken rock. Treatment of the weak rock added to the difficulties encountered in creating an effective water barrier by pressure grouting.

One of the early developments to grout a foundation more effectively was the well known stage method. This has been well described by others and will

not be amplified here.[3] A further development was the "zone" method[2] whereby a whole section is stage drilled and grouted to a limited depth before any holes are drilled and grouted beyond that depth. This further protected against foundation displacement and excessive surface leakage. The "closure" or "split spacing" method[2] as described by others[3] also reduced somewhat the danger of excessive displacement as well as reducing time and cost by keeping the footage of drilled holes to a minimum. However, even with these developments, excessive surface leakage and near surface "lifting" occurred too often, especially in the weaker rock foundations. To further minimize the danger of movement and excessive surface leakage the use of a packer was adopted in certain areas. Its use was so effective that it soon became standard practice on many types of foundations.

Description and Use

Packer (also called "stop") grouting is simply the technique whereby a section of a drilled hole is grouted separately by setting a "packer" at the desired location in the hole and grouting that portion below the packer. The packer is then raised a predetermined distance and the new section grouted, usually at a lower pressure. This procedure is repeated until the entire hole has been grouted. Most holes in an average foundation can be drilled to their full depth initially and grouted in the above manner. Conditions do arise, however, that cause or warrant a modification of the procedure to obtain better results. Distance between settings, pressures used, and other details vary with foundation materials and conditions and will be discussed later.

There are three general types of grout packers in common use, the cup leather, the mechanically expanded rubber ring, and the pneumatically expanded rubber sleeve. Various methods of mechanically expanding the rubber ring or rings have been used and each has its place for a particular condition. No effort will be made to illustrate all of the packers that have been used as all types are frequently modified and improved to fit local conditions. Each general type does, however, have characteristics making it most suitable to rock types. Initially, Ax(2") or Bx(2-3/8") holes were thought to be the smallest size adaptable to the use of packers, but now they have been developed for all sizes from Ex(1-1/2") to Nx(3"). Some difficulties arise when the smaller LM(1-3/8") hole is used. The packers to be described here are all for Ex size hole.

The cup leather type shown in Figure 1(a) is best suited to fairly hard rock where the drilled hole is not oversize and the walls are relatively smooth and true. This packer when suitably anchored has been used successfully for grouting pressures up to 750 psi. It is simple to construct, easy to maintain, and only requires a single pipe to lower it in the hole. Where high grout pressures are feasible, it is probably the best type of packer to use. If it should accidentally become stuck in the hole, a "right-left" coupling enables the crew to save the supply pipe string and the packer itself can be drilled out, if need be.

The mechanically expanded type shown in Figure 1(b) is adapted to somewhat poorer rock than the cup leather type, but it may be difficult to seat if

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3. See Selected References.

the drill hole is too much oversize. Its positive expanding action gives it an advantage in that it can be positively seated at any location if the hole is not too enlarged. When used at depths greater than 20 feet, flush-joint tubing is required and it is somewhat awkward to handle in a deep hole. Once seated it too can withstand fairly high pressures and has been used on many jobs.

The pneumatic packer shown in Figure 2 has proved suitable in poor rock where the drill holes are quite oversize. In fact the Ex(1-1/2") size can be seated in a 4-inch pipe when the proper rubber tubing is used, and it is properly attached at the ends. The length of rubber sleeve should not be less than 18 inches. Under conditions requiring large expansion and relatively high expanding pressure, double-tapered collars at either end may be necessary to prevent rubber breakage. It is not suitable for high grout pressures, but it will withstand 100 psi under poor conditions and will hold up to 200 psi if the hole is not too large or uneven. In weak sedimentary formations of alternating layers of shale and sandstone or limestone, this packer has proved invaluable. It is now widely used where low pressures are dictated by foundation conditions.

The distance between packer settings will vary with local conditions and is determined by trial on the job. Except in special cases, the distance is not less than 10 or 15 feet in the upper section of a hole. The lower section of a deep hole may not require a packer setting closer than 50 feet or more from the bottom. The usual distance between packer settings is that which will allow the desired pressure to be maintained by the grout pump. In this way each section of the hole is being utilized to its maximum. Where a special feature is encountered, it can be isolated and treated as required without losing the value of the remainder of the hole. A special case develops where there is loss of the drill water. To avoid the danger of choking the opening with drill cuttings, drilling is discontinued, a packer is seated just above the point of loss, and the joint or channel is grouted. In this way an open channel can be treated to advantage without regard to the rest of the hole. Encountering artesian water may be handled in a like manner, depending on the amount of flow. If the flow is not too large, it can be treated separately by recording the depth at which it was encountered and seating the packer just below and then just above as the hole is grouted from the bottom up.

In some instances it has been deemed desirable to combine packer and stage drilling and grouting. This practice may be warranted in a particularly weak foundation material but is avoided unless there is definite evidence of an advantage to the result. Consistent surface leakage from grouting the deep section of the holes would warrant this procedure. Another case where it is necessary to drill in stages is where caving ground is encountered. By seating a packer above the caved section it is often possible to "grout off the cave" and the upper section of the hole will not be fouled. In the interest of economy, however, where packer grouting is planned, stage drilling should be held to a minimum and used only where required.

The pressures used in packer grouting are more readily established than when grouting without a packer. The safe allowable pressure is determined by the depth of cover over the packer and will vary from as low as 3/4 of a psi per foot to as much as 2 or 2-1/2 psi per foot of cover, depending on the unit weight of the material and the geologic structure. In good rock a constant factor may be added. Particularly where the higher pressures are being used, care must still be exercised and it may be necessary to reduce the pressures, especially on holes that take grout rapidly. Where an adjacent hole returns

grout in a sufficient amount to require closing, a packer should be set in the returning hole just above the point of inflow in order to prevent excessive pressure at the surface. Establishing the pressures to be used in packer grouting still requires experienced judgment, but due to the rock cover above the packer setting, higher pressures can be used with safety.

There are several advantages and some disadvantages in packer grouting. One of the biggest advantages is the ability to apply higher grout pressures to the deeper portion of the grout holes without increasing the danger of near surface movement. This often allows for greater spacing between deep holes with the resultant saving in time and effort. Another advantage is the definite information obtained as to what depths and in what amounts the various depths of a foundation take grout. This is important in determining the amount of grouting required and often saves much drilling effort, or on the other hand, pinpoints the need for additional grouting in a particularly weak zone. Stage drilling and grouting methods provide similar information but with much less accuracy as there is always the possibility, and often the probability, that the upper part of the foundation has displaced and the grout may not be going into deep section of the hole being stage grouted.

A third distinct advantage is that the danger of excessive surface leakage is minimized. Not that grout being injected at depth cannot find its way to the surface; it sometimes does; but there is much less chance of its doing so. In some cases where it does come to the surface, much of the cement has had an opportunity to settle out and be retained in the foundation, and the leakage is largely water. Eventually, of course, the cement loss may become excessive but the potential water course should be pretty well filled by that time.

A major advantage from the construction standpoint is the saving in time of drilling and grouting the foundation. The time saved by not having to move the drill from the hole until it is completed and having to more the grout lines from the pump to the hole only once is often of major importance. This saving may not be gained on all of the holes as some of them will still have to be stage drilled for various reasons, some of which have been described above, but under normal conditions the majority of the holes will be completed in one setup.

There are also some disadvantages to packer grouting. One of them is the extra equipment required, the packer itself, and the pipe or tubing needed to place it in the hole. The packer rubber or leathers will wear and need replacement and occasionally a packer assembly may be lost. The pipe or tubing will last out the job if properly cared for. However, the rubber tubing or cup leathers are not expensive, and the entire assembly is often shop made.

The time required to seat the packer may be listed as a disadvantage, although this takes only a fraction of the time required to move a drill. When a crew becomes familiar with the procedure, it only requires a few minutes to seat a packer. Sometimes it is not possible to seat the packer exactly where planned, but it can nearly always be set within 5 feet of the spot.

There are those that contend that the elimination of repeated grout applications to the upper sections of a hole is a disadvantage. This may be true in certain types of rock that will accept grout even after having been grouted once or twice previously. This type of grouting has been called multiple grouting. However, this condition is not encountered often. Experience shows that grout take in a previously grouted hole is usually the result of foundation displacement or the reopening of a previous surface leak caused by the application of the higher pressure.

There are other minor advantages, and possibly disadvantages, that space will not permit listing here, but the above point out the major ones. The increasing use of packer grouting by design engineers and its increasing popularity with construction organizations would indicate that the advantages outweigh the disadvantages. Most drilling and grouting specifications now contain provisions for packer grouting as well as stage methods.

An opportunity to observe stage grouting and packer grouting in the same area was recently provided at an earthfill dam site. The foundation material consisted of a chalk formation overlying an impervious shale. The chalk contained fairly large, near vertical cracks, especially frequent adjacent to the valley slopes. It was further weakened by thin, horizontal shaley layers. Grout holes drilled at a 60° dip to cross the vertical cracking were extremely irregular in diameter and suitable pneumatic packers were not initially available. The first grouting was done by the stage method using pressures of 10 psi or less. The holes accepted grout rapidly where they crossed an open crack and thick grout was used. After several holes had been grouted and large quantities of cement had been used, it became apparent that the foundation was being lifted. Apparently, when the open cracks became full or nearly full and as pressure was applied, the weak shaley seams split, causing the chalk above them to lift. Grouting was discontinued until suitable pneumatic packers could be obtained, and the remainder of the grout curtain was created using packer grouting methods. Although the grout take was still quite high in certain areas, there was little or no lifting subsequent to the the use of the packers. The above may be an extreme case, but it illustrates a condition where use of packers to grout the formation was almost necessary to obtain a good result. In addition there was a considerable monetary saving.

Another example of the beneficial use of packer grouting was shown during the grouting of the primary curtain under a large concrete dam. Grout at 400 psi was being injected through a packer placed about 75 feet below the collar of the hole. (The dam was in place.) Although the rate of take was relatively low, a previously grouted hole 10 feet away returned grout. A valve was placed on the collar pipe of the returning hole. Soon after this valve was closed, the instruments placed to observe for movement gave their warning. Subsequently, whenever an adjacent hole returned grout, a packer was seated just above the point of inflow, thereby minimizing the danger of high grout pressures being applied to the upper section of the foundation. Where a packer could not be seated, the pressure was reduced to the same that was used at the collar of the hole. No further movement was detected on the entire job. It is probable that grouting without the use of packers may have caused considerable movement with the accompanying increase of cost, and, if excessive, damage.

There are many other examples of the benefits of packers in pressure grouting, but the above two are of extremes of poor and good rock foundation material. They show the wide range of the possible use of packer grouting.

CONCLUSION

One of the more serious problems encountered in pressure grouting most foundations is the danger of excessive rock movement. By positively controlling the point of injection in the deeper sections of the grout holes with packers, that danger has been materially reduced. Where packers are being

used, it is not necessary to apply high grout pressures to the top sections of the grout holes while still allowing the use of higher pressure in the deeper sections. The procedure allows weaker foundation rock to be effectively grouted and reduces the number of deep holes required in grouting a strong rock.

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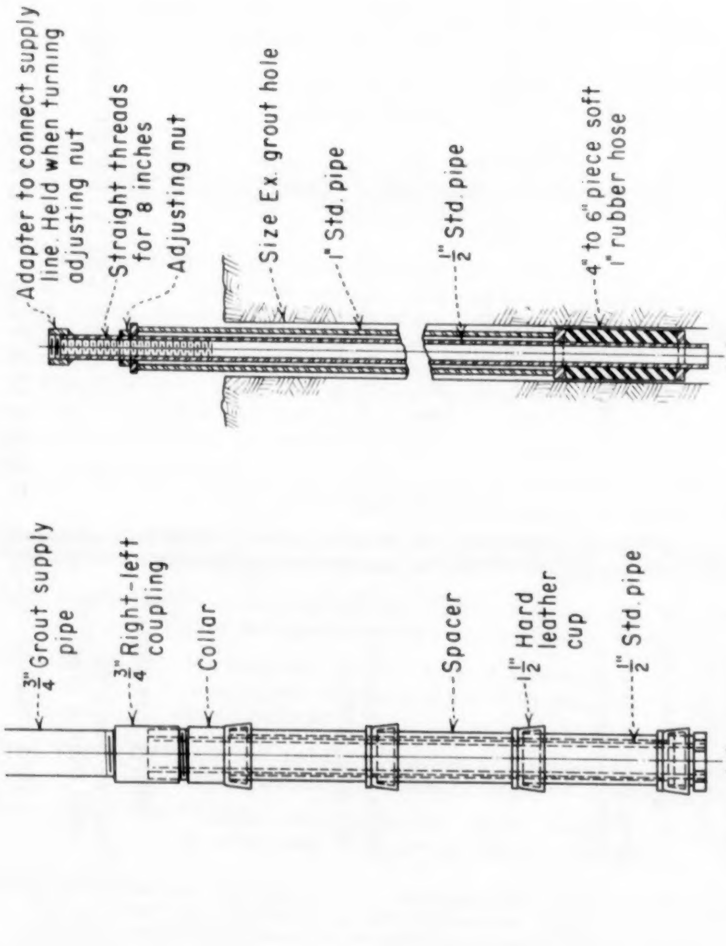

CUP LEATHER PACKER MECHANICAL PACKER

FIGURE 1

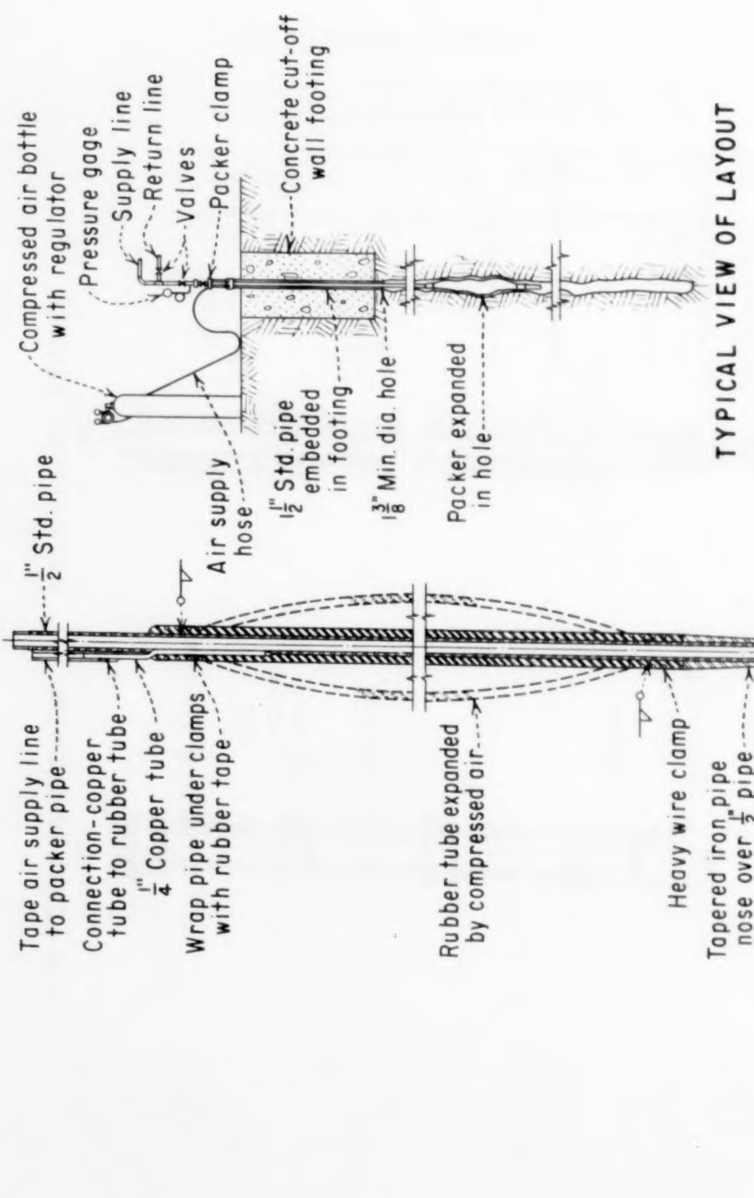

FIGURE 2

PNEUMATIC PACKER

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Proceedings of the American Society of Civil Engineers

CEMENT AND CLAY GROUTING OF FOUNDATIONS: FRENCH GROUTING PRACTICE

Armand Mayer,[1] M. ASCE
(Proc. Paper 1550)

FOREWORD[a]

The work of the Committee on Grouting of the Soil Mechanics and Foundations Division is divided among four task committees, concerned with (1) bituminous, (2) chemical, (3) cement and (4) clay grouting.

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1. Cons. Engr., Chairman of the French Cement-Research Organization, Paris, France.

a. By Raymond E. Davis, Chairman, Committee on Grouting.

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The information contained in the papers has been assembled for the particular benefit of those responsible for the foundation treatment of structures. Recognizing that these papers by no means represent the "last word" in good practice and that there may be many others concerned with foundations who could contribute to our knowledge of cement and clay grouting, discussion is earnestly sought.

ABSTRACT

European experience has indicated that clay and clay cement grouts can be utilized to successfully control seepage through alluvial materials. Descriptions of foundation conditions, grouting programs and nature of the grout mix are given for four projects: Genissiat cofferdam, Fessenheim open pit, Ait Ouarda cofferdam, and Serre-Poncon dam.

INTRODUCTION

The use of cement grouts for stopping leaks in fissured rock has been known for many decades. Yet, previous to 1925 all attempts to make a pervious sand or gravel layer watertight by grouting had failed.

Joosten was the first to succeed about 1925, by using two chemicals, sodium silicate and calcium chloride, which when mixed, produce a gel. Joosten used a double grouting pipe with which he injected separately both products. These formed a gel as soon as they came in contact. The gel filled the voids of the pervious layer and made it watertight. The process worked well, but as the gellifying process was instantaneous, grout holes had to be drilled 1' apart in order to obtain a continuous cut-off.

A very distinct improvement was obtained with the single pipe grouting process. By replacing calcium chloride by other chemicals, grouts could be obtained which turned to gels after given periods, say 1/4 or 1/2 hour. By timing the mix properly grouts could be prepared which gellified after having infiltrated the proper distance from the grout hole. The only limit to the distance two grout holes could be placed apart was the time necessary to fill with the grout the voids located between these holes. The first practical use of the single pipe grouting method was the cut-off in Bon Hanifia (Algeria)

which was carried out under the technical guidance of Prof. Terzaghi about 1933.

But sodium silicate was expensive and the method remained restricted to very special cases.

Some time later the Paris soil mechanics laboratory started improving the method by mixing a cheaper material with the silica grout. Different types of fine grained soil were tested, ranging from medium silt to bentonitic clays and the tests showed that a stable grout of progressively increasing viscosity could be obtained by adding just a small proportion of silicate mix to the clay suspension. This improvement not only made the process more useful, but at the same time made it much more economical. It was used on a large scale for grouting the upstream cofferdam of the Genissiat dam and several other projects. A short description of the work as carried out in Genissiat in 1937 and 1938 will be given hereafter.

The apparent permanence of an impervious curtain after 7 years of experience encouraged Electricite de France to agree to the use of grouting in different cases where a curtain had to be constructed through alluvial materials. But the question was not yet completely solved. How long would such a screen hold out under water pressure? In order to have a permanent stability, the engineers of Electricite de France requested that a certain proportion of cement be added to the mix in order to have an irreversible set. This was the incentive for the clay-cement grouts which have now become common practice in France.

The first tests on premixed alluvial materials failed completely. Cement alone, even when finely ground, just did not penetrate the sand and gravel. Cement mixed with stabilized fine clay behaved somewhat better but still penetrated only quite coarse materials. It was not until the very complete studies of the alluvial materials of the Durance River for Serre-Poncon and of the Rhine for Fessenheim showed the high permeability of the layered heterogenous natural materials that the question was taken up again. The tests in the field showed that even if the cement did not penetrate the layers of fine sand, the results achieved in the open gravel and the coarse sand were sufficient to yield an all round permeability of the order of 10^{-5} cm/sec., sufficiently low so as to practically suppress all leakage.

So it came that what had been considered as a failure in the laboratory happened to be a success in the field. Two improvements also helped in obtaining good results; the high velocity mixers now used by practically all good grouting contractors which change the granular composition of the cement and increase the stability of the grouts. Also the finely ground cement, as obtained by the slag wet grinding process. The wet grinding, which can only be applied to slag before any catalyser has been added to it, reduces the necessary grinding power and produces a finer cement than the ordinary dry operating method.

Since then, laboratory studies have been carried out and have shown the influence of both the grain size and the rheologic characteristics of the grouts. These studies have made it possible to predict the performance of the different grouts in a given material. They have proved that clay-cement grouting can be used in many cases but not in all. The author has come across several examples where the medium was much too fine to be grouted with cement-clay. The author was called upon to investigate a large river bed in India, for instance, which was filled with an homogenous fine sand which can be grouted with chemicals only. The object of this paper is to present

four cases where clay and clay cement have been used successfully and to show the conditions that existed on the different projects.

These projects are:

The Genissiat upstream cofferdam grouted in 1937-1938
The Fessenheim pit excavated in 1951
The Ait Ouarda cofferdam, also grouted around 1950
The Serre Poncon cut-off started in 1954 and actually completed.

Genissiat 1937-1938

It is now almost 20 years ago that, on the author's suggestion, the Compagnie Nationale du Rhône agreed to carry out the cut-off of the upstream cofferdam of the Genissiat structure by using the process of clay grouting that had been tested only few months before on the Sautet Project. The process and the results have been described in a report presented at the 1948 meeting of the Large Dams Conference. Only a summary of the more significant factors will be repeated here.

In the vicinity of Genissiat between Lyon and Geneve the Rhone River flows in a limestone canyon filled to a depth of 25 meters by alluvial materials. The dam being a gravity dam, the design called for an excavation down to the limestone which had to be kept dry during the whole construction period. In order to avoid excessive pumping a sheet pile cut off down to the rock had been designed under both the upstream and downstream cofferdams. When it came to the construction, tests showed that driving the sheetpiles for the upstream cofferdam was impossible. Heavy boulders were encountered in the sand and gravel which no sheet pile could break apart. Grouting tests with cement, which at that time was the only common grouting material, did not succeed in making a watertight curtain through the alluvial strata. A suggestion to use both chemicals and stabilized clay was accepted by the Company and carried out successfully. Laboratory tests showed that while the sodium silicate could penetrate the fine sand layers, the clay could stop the leakage in the open gravel and coarse sand. It was therefore decided to drill three parallel lines of holes and to grout the outer ones with clay, and the middle one with sodium silicate. A slow setting sodium silicate and hydrochloric acid mix was added to the clay in order to increase the thixotropic character of the clay suspension. As a result the clay grout thickened after having been pumped in, and no water pressure could leach it out. The way the clay had penetrated the sand and gravel was shown during the drilling and grouting of the intermediate holes. The water used for drilling came out of the hole quite white, carrying a high proportion of clay. The amount of silicate grout that could be pumped in just corresponded to the volume of voids between the two exterior lines. No leakage could be detected and in all the intermediate drill-holes, the grout could be pumped in under a pressure as high as 10 kg/cm2 (140 psi), higher than the water pressure after construction of the 80 meters high dam.

The only difficulties encountered were due to the torrential flow of the Rhone River, the level of which several times changed by 2 or 3 meters in a night. The first time it happened, the drilling-rig broke loose and was carried several miles down the river. Care was afterwards taken to prevent such accidents. The result was excellent. The excavation could be pumped dry easily. More over the effectiveness of the grouted curtain proved to be

quite permanent. The construction was interrupted by the war and for several years the excavation remained flooded. When the war ended and work was again started, it was found that the excavation could be unwatered with no more difficulty than before.

Pumping could even be stopped on the upstream side, where the cutoff had been obtained by grouting, whereas it had to be continued permanently on the downstream side where sheetpiles had been used.

Following figures will give an idea of the magnitude of the work:

Surface of the cut-off: 300 m2 = 3000 sq. ft.

Materials used	(634 T. clay
	(10 m3 silicate of sodium
for the clay grout	(7 m3 hydrochloric acid (1.16 density acid, diluted 1/10)
for the silicate grout	(44 m3 silicate
	(3.8 m3 acid.

Fessenheim 1950 - Vogelgrum-Marckolsheim 1956

The second example consists of three projects, Fessenheim, which is now practically finished, Vogelgrum and Marckolsheim, several miles further downstream, when the curtain grouting has been completed and excavation is advancing rapidly. The conditions at the three sites are similar.

All are located in the Rhine Valley on the stretch between Strasbourg and Basel, where the river level drops by 100 m. in less than 100 km. They are parts of a large power-navigation project intended to make the navigation along this stretch easier even by low waters and to use the drop for power. Two dams and locks had already been built before, at Kembs and Ottmarsheim near Basel. The corresponding locations were on rock so that no foundation difficulties had been encountered. But downstream from Ottmarsheim the bedrock dips more than 200 m. so that the more recent projects had to be set on an alluvial base, with a waterlevel almost at the ground surface. As the Vogelgrum-Marckolsheim projects are still in the completion stage, the figures given hereafter will apply to Fessenheim only.

The problem there was to dig two open pits, one for the power plant 143 x 87 m (420 x 270') large, 23 m (60') under the water level, the other 185 x 23 m (540 x 60') large for a lock with an 18 m. (54') raise. The preliminary borings showed that the materials in which the pits had to be dug were layered and very heterogenous ranging from fine sand to open gravel, with an average permeability of 4×10^{-1} cm/sec. It was therefore decided to enclose the pits in a box made of grouted horizontal and vertical curtains to cut off the flow.

Preliminary tests showed that the alluvial materials in situ could be divided in two categories: the fine sand that could only be grouted with chemicals, and the coarse sand and open gravel, that could take cement-clay grouts. It was therefore decided that for every drill hole, a boring profile would be made showing the different strata to be used as a guide during the grouting process. Tests also showed that the grout holes could be 3 m. apart for the vertical curtains while they could be 9 m. from each other for the horizontal curtain. Finally measurements on a three-dimensional model made it possible to determine at what level the horizontal screen should be placed so as to cut off the flow without producing any uplift of the bottom of the pit.

After having completed all these preliminary studies the work started at a very rapid pace.

The grouts used were:

in the coarse sand and open gravel a mix of: cement 20 kg
clay 70-95 kg
water 100 liters

in the fine sand a mix of: sodium silicate 30 liters
sodium aluminate (diluted) 25 liters
water 30 liters

The materials used amount to 35,000 tons of clay, 5,500 tons of cement, 2,000 tons of chemicals, and 22,500 m. bore holes were drilled.

The average permeability of the alluvial material was reduced from 4 x 10^{-1} to 10^{-4} cm/sec, so that the total seepage, instead of 6 m3/sec as it would have been, was reduced to 115 liters/sec. The result was considered very satisfactory and the same design will be adopted in the projects now in the construction stage.[1]

Ait Quarda 1950

The Ait Quarda project, carried out in Morocco in 1950 and 1951, is an example of a cut-off under a cofferdam, very similar to that of Genissiat. The Ait Quarda dam is a gravity structure, built in a valley filled with 20 m. alluvium over limestone. In order to excavate down to the limestone, it was considered necessary to keep the excavation dry. 7 m. sheet piles, recovered from a previous job, were available near the site. So it was decided to use these for the 7 first meters below the cofferdam and to close the remaining depth between the sheet piles and the underlying rock by means of a grout curtain.

The alluvium consisted of several lenses of clay, separated by layers of coarse sand of a diameter of more than 0.5 mm. The available clay was a material finer than 0.02 mm. The relation between the grain size of the grout and that of the alluvium was around 25, which is a favorable one. A high velocity mixer was used so as to obtain a more stable grout and to reduce the diameter of the cement particles added to the mix. Laboratory tests showed that proportion of cement had to be added so as to have a cohesion considered sufficient by the engineer. They are summarized hereafter:

$\frac{\text{Cement}}{\text{Cement + clay}}$ ratio	Compressive strength in kg/cm2 after 7 days	:	after 28 days
20	0.4	:	5
30	1	:	7
40	5	:	8

A percentage of 20% cement/cement + clay was considered sufficient to obtain a permanent curtain. The final composition of the grout per liter was:

Clay	310 gr.
Sodium silicate	11 gr.
Cement	78 gr.
Water	850 gr.

1. A complete description of the job at Fessenheim was given by Mr. Lefoulon and Mr. Ischy in a report to the Paris Large Dams Meeting in 1955.

The work was carried out with excellent results. The leakage through the alluvial materials was stopped altogether.

Serre-Poncon 1954-1956

The grouting of the foregoing projects was in the nature of a construction expedient and it was neither required nor planned that the grout curtain should remain effective beyond the construction period. Nevertheless at Genissiat, the curtain was still impervious after the delay of 7 years due to the war.

At Serre-Poncon the possibility of cutting off the flow in alluvial materials under the dam permanently and to a depth of 100 m. was considered by Electricite de France as the necessary condition for the construction of this 120 m. high earthdam. This condition was introduced more for morale purposes than because of a technical requirement. It was the first high earth dam to be designed in France, the amount of water in the reservoir was considerable and the valley, downstream of the dam, a very populated area. Before the project was approved by Parliament and incorporated in the program of Electricite de France, the fears of local politicians had to be calmed and this would not have been possible if it had been known that water was still flowing under the dam as it might have done without any danger to the structure.

The problem here was not only to make a cut off, but to make it permanent and also to have it sulfate resistant. Mineral water springs had been discovered flowing out of faults in the bed rock underlying the alluvial blanket. The water from these springs had a high sulfate content due to the presence of important gypsum deposits upstream from the site. Electricite de France therefore decided that the grout should contain a certain proportion of cement that would set and prevent any leaching of the cut-off, and that instead of Portland, sulfate resisting cement would be used, with sodium hydroxide as a catalyser. Part of the equipment that had been used at the Bort dam for wet-grinding the slag was to be shifted to Serre-Poncon. The finely powdered slag had a specific surface, as measured with the Blaine testing equipment, of 4.000 cm2/g as against 2500 for ordinary Portland Cement, which means it had the grain-size of a silt. It was perfectly sulfate resistant. Conditions were therefore as favorable as possible for obtaining a cemented cut-off.

Grouting tests were made in the laboratory on the finer elements of the alluvial materials. For this purpose a clay-cement grout was used containing about 30% cement. The maximum grain-size of the cement was 0.04 mm and the minimum grain-size of the alluvial materials was approximately 0.1 mm., a ratio of minimum soil size to maximum grout size of 2.5. This value was too low to permit the cement to penetrate the fine grained materials. The boring tests had shown that the deposit consisted of an alternating series of fine and coarse layers. The minimum grain-size of the coarse layers was 1 mm. which gave a soil-grout ratio of 25. The laboratory tests showed that with this ratio excellent grout penetration could be accomplished. Consequently it was decided to grout the coarse layers with a clay-cement grout and the fine-grained layers with sodium silicate. In some cases the fine grained layers were intermediate in grain size between the finest and the coarse layers. In these materials a clay-sodium silicate grout was used for reasons of economy.

The grouting of the alluvial material at Serre-Poncon has been a tailor-made job, the packers being only 30 cm high so that the nature of the grout could be changed, if necessary, 3 times in a metors height. The fact that slag was used instead of cement was also excellent, as a slag mix can be kept practically indefinitely without setting under normal temperature and pressure conditions. It sets only when mixed with a catalyser. Sodium hydroxide was used in Serre-Poncon in a proportion of 0.5% of the cement and added at the last moment just before grouting. A description of the packers used at Serre-Poncon will be found in Mr. Maigrets report to the 1955 Large Dams Conference. They are of the type called "sleeve packers." They make it possible to change the grout as often as necessary and to grout the same layer several times, if required, without building up an impervious curtain around the drill hole that would stop all grouting. Operations are as follows: first drilling the hole, generally mud-drilling, sometimes also water drilling. Then a perforated pipe is lowered into the hole. The holes in the pipe are generally every 30 cm. apart, covered with a rubber sleeve which, under pressure expands and lets the grout flow. The hole is then washed out to remove the mud. Then the interval between the pipe and the soil is filled with a low-resistance cement mortar. Finally the injection pipe with the perforated packer is lowered into the bore-hole. When grout is pumped in, the rubber sleeves expand, crack the outer mortar, and let the grout flow into the soil.

This method has been used at first in order to make a test pit and to allow for direct inspection of the injected soil. A square area was selected in the river bed and grouted all around. Next a circular well was sunk in the middle and made ready for direct inspection. This well, which was shown to the members of the 1955 Large Dams Conference, demonstrated that it was possible to get down 40 m under the waterlevel without any visible waterflow along the walls of the well. The experience was so successful that it was decided to carry on and to continue the same method all thru the river bed. The work is now completed and construction of the dam is moving fast. The diversion tunnels are finished, the base of the dam has been laid dry, the cut-off trench having been dug over the grouted section of the valley. The digging was stopped when the trench reached the grout. Then it was filled with impervious materials and compacted according to the specifications for the impervious core. That way it will be quite certain that an impervious continuous screen will have been built across the valley.

As construction is still going on no final figures can be given yet. The data for the test section were as follows:

grout into rock (10 m. deep)
- grouting pressure from 10 to 60 kg kg/cm^2
- absorption: 600 kg/meter drill hole

contact between the rock and the alluvium
- very permeable zone - grout: 1/3 cement, 2/3 clay.
  absorption 4.4 t/meter drill hole.

alluvium
- bore holes every 3 m. The grout was first made every second hole and in every hole every 33 cm.
- Before grouting the layer was tested with water.

Where the material was very pervious it was grouted with a mix of clay and slag (with 0.5% sodium hydroxide). The permeability was thus reduced 100 times. If the material was only little pervious, it was grouted with a

mix of clay and chemicals, which reduced the permeability 500 times.

The general conclusions drawn from the tests are:

1. It is completely practicable to make an impervious curtain through layered, very pervious, alluvial materials such as those at Serre-Poncon. The average permeability in the test sections has been reduced from 0.5 to 1 x 10^{-1} to 1 x 10^{-4} cl/sec. The average consumption of grout was of 400 kg/m3 treated material.

2. The waterproofing effect is better with the clay-silicate mix than when using cement. In no case should a neat cement grout be used. When using cement-clay grout, the proportion of clay should be at least 2/3 of the mix.

3. Grouting shallow layers is impossible. A certain weight is necessary to counteract the effect of pressure grouting. No result can be expected in the first 5 - 7 meters under the surface.

4. Special care should always be taken at the contact zone between the rock and the alluvium. The consumption of grout in that zone can be 10 times the average.

Paper 1551 February, 1958 SM 1

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Proceedings of the American Society of Civil Engineers

CEMENT AND CLAY GROUTING OF FOUNDATIONS: PRACTICE OF THE CORPS OF ENGINEERS

Edward B. Burwell, Jr.,[1] M. ASCE
(Proc. Paper 1551)

FOREWORD[a]

The work of the Committee on Grouting of the Soil Mechanics and Foundations Division is divided among four task committees, concerned with (1) bituminous, (2) chemical, (3) cement and (4) clay grouting.

The Task Committee on Chemical Grouting has completed its task and its report was published in the Division Journal, November 1957.

"The Symposium on Cement and Clay Grouting of Foundations" is a "joint venture" of the Task Committee on Cement Grouting and of the Task Committee on Clay Grouting. It includes the following papers:

Proc. Paper 1544 "Cement and Clay Grouting of Foundations: Present Status of Pressure Grouting Foundations" by A. Warren Simonds

Proc. Paper 1545 "Cement and Clay Grouting of Foundations: Grouting with Clay-Cement Grouts by Stanley J. Johnson

Proc. Paper 1546 "Cement and Clay Grouting of Foundations: The Use of Clay in Pressure Grouting" by Glebe A. Kravetz

Proc. Paper 1547 "Cement and Clay Grouting of Foundations: The Use of Admixtures in Cement Grouts" by Alexander Klein and Milos Polivka

Proc. Paper 1548 "Cement and Clay Grouting of Foundations: Suggested Specifications for Pressure Grouting" by Judson P. Elston

Proc. Paper 1549 "Cement and Clay Grouting of Foundations: Pressure Grouting with Packers" by Fred H. Lippold

Note: Discussion open until July 1, 1958. A postponement of this closing date can be obtained by writing to the ASCE Manager of Technical Publications. Paper 1551 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SM 1, February, 1958.

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a. By Raymond E. Davis, Chairman, Committee on Grouting.

Proc. Paper 1550 "Cement and Clay Grouting of Foundations: French Grouting Practice" by Armand Mayer

Proc. Paper 1551 "Cement and Clay Grouting of Foundations: Practice of the Corps of Engineers" by Edward P. Burwell, Jr.

Proc. Paper 1552 "Cement and Clay Grouting of Foundations: Experience of TVA with Clay-Cement and Related Grouts" by George K. Leonard and Leland M. Grant

These papers are based largely on the results of successful grouting operations for a wide variety of foundation conditions but also are based on the results of laboratory investigations and to some extent on the ideas and opinions of designing engineers who are concerned with foundation problems.

The information contained in the papers has been assembled for the particular benefit of those responsible for the foundation treatment of structures. Recognizing that these papers by no means represent the "last word" in good practice and that there may be many others concerned with foundations who could contribute to our knowledge of cement and clay grouting, discussion is earnestly sought.

SYNOPSIS

Cement grouting practice has not attained a degree of standardization commensurate with other construction practices because of the human factors and the wide range of variables involved. As high a degree of standardization as is practicable, based on a comprehensive study of the impressive accumulation of experience and information on cement grouting, is needed in the interest of economy and efficiency and to reduce to a minimum the intangibles of cement grouting specifications. This paper, presenting the cement grouting practice of the Corps of Engineers, has been prepared as an aid in the current study of the problem by the ASCE Committee on Cement Grouting of Rock Foundations.

INTRODUCTION

Cement grouting is the most important and widely used method available to the construction industry for reducing the mass permeability of bedrock. However, in spite of its universal application as a means of treating dam foundations, cement grouting practice has failed to attain a degree of standardization commensurate with other construction practices.

Principal among the influences that have retarded standardization are (1) the atmosphere of mystery in which the art developed, (2) individual prejudices, (3) the difficulty of establishing definite procedures and rules for a method of treatment involving so wide a range of variable factors and (4) the controversial nature of the art itself. Indeed, for a long time, the art was so shrouded in mystery and veiled in nebulous techniques that its practice was left largely in the hands of a rare group of individuals who, for one reason or another, had come to be regarded as "grouting experts." As a result, the

pattern of design and procedure into which a grouting program fell generally bore the hallmark of the individual to whom the program was assigned.

While a lack of unanimity still exists with respect to some of the fundamental principles and procedures of cement grouting, the obscuring influences have been dispelled to a large extent by the common knowledge and practical experience that have grown out of the vast amount of successful grouting accomplished in the last two decades.

It is unlikely that some of the more controversial matters of design and procedure can be fully resolved at this time because individual experience is a prime factor in this problem. Moreover, case histories show that the objectives sought in foundation grouting have been attained effectively by the use of widely different methods and techniques. For example; most of the curtain grouting of the Corps of Engineers has been done by the use of a single line of grout holes and by the stage method. In contract, there are those of wide experience who use multiple lines of grout holes, who regard blanket grouting as a basic phase of a curtain grouting program, and who consider the packer grouting method generally preferable to the stage method.

What appears to be needed most now is a composite sampling of the impressive accumulation of information and experience with a view of establishing as high a degree of standardization of cement grouting practice as is practicable in the interest of economy and efficiency and to reduce to a minimum the intangibles of cement grouting specifications.

Some ten years ago the Corps of Engineers issued, in preliminary form, certain manuals and guide specifications, based largely on its own experience, for the use of its field installations in planning and executing grouting programs. These have been revised from time to time to include such new developments in equipment, materials, and techniques as field experience and research demonstrated to be desirable. As a result, the Corps' grouting practice during the last decade has adhered rather closely to a reasonably well established pattern of design and procedure.

It is the purpose of this paper to outline and discuss in broad terms the cement grouting practice of the Corps of Engineers and its current thinking with respect to specifications requirements. It is hoped that discussions will be invoked that will be helpful, not only to the Corps in its current undertaking of revising the manual and guide specification on foundation grouting, but also to the ASCE committee on Cement Grouting of Rock Foundations in preparing its final report.

Types of Treatment

Cement grouting of rock foundations, according to the Corps' terminology and practice falls into five classes: curtain grouting, consolidation grouting, contact grouting, dental treatment, and slush grouting.

Curtain Grouting

Curtain grouting is defined as the drilling and grouting of one or more lines of holes in a foundation or reservoir rim for the purpose of creating a barrier or cutoff against excessive seepage.

Virtually all of the curtain grouting done by the Corps in the last ten years, and most of the curtain grouting done prior to that period, has been accomplished by the use of a single line of grout holes.

For concrete dams, the curtain grouting is done from a grouting and drainage gallery after the concrete has reached either full height or a height that will permit the use of the maximum working pressure without endangering the structure or its foundation.

For earth or rockfill dams, the curtain grouting is generally done before any fill is placed, using a single line of grout holes located along the centerline of the core and drilled from the surface of bedrock or the bottom of a cutoff trench as the case may be. The Corps' experience in drilling and grouting through embankment fill generally has proven unsatisfactory for the following reasons: (1) pressure washing and pressure testing operations cannot be effectively accomplished because of the soil erosion problem, (2) gage pressures have to be limited to a few pounds per square inch because of the small differential in unit weight that exists between the grout slurry and the embankment fill and (3) grouting through the fill increased the costs materially.

The Corps' practice of using a single line of grout holes for curtain construction has produced excellent performance records under a wide variety of foundation conditions and under reservoir heads ranging up to 400 feet. The Corps' principal objections to the multiple line method of construction are (1) it requires a substantially larger footage of grout hole drilling, (2) for concrete dams, all lines except the gallery line must be drilled and grouted in the open where interference and delays to other construction operations may result, (3) if the multiple line design includes a line of holes downstream from the gallery line, the drainage system may be made less effective, and (4) grouting of the outside lines of holes will generally have to be done at low pressures to avoid upheaval, since this grouting is done either from the foundation surface or through a relatively light weight of concrete.

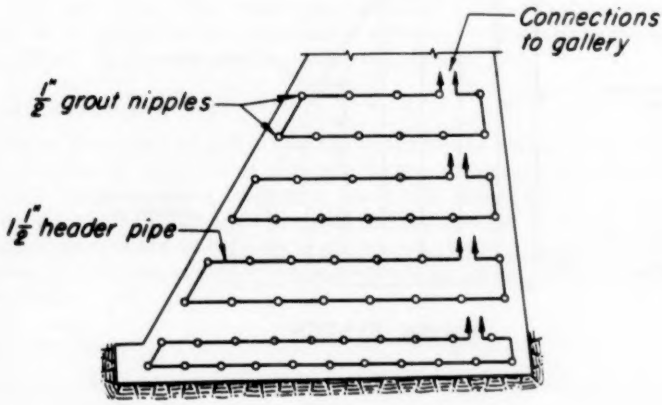
Consolidation Grouting

Consolidation grouting, frequently referred to as blanket grouting or area grouting, is defined as the drilling and grouting of a pattern or grid of comparatively shallow holes at relatively low pressure for the purpose of consolidating a mass of highly fractured or otherwise defective rock. It may be done to fortify the curtain grouting, to improve the foundation or to reduce the depth of excavation. The Corps has made only limited use of consolidation grouting and generally does not favor it as a means of fortifying the curtain grouting or reducing the depth of excavation. Its principal use by the Corps has been either for the purpose of taking up the slack in highly fractured but otherwise acceptable foundation rock or to remove and replace with grout objectionable materials in foundations such as the clay fillings in solution channels.

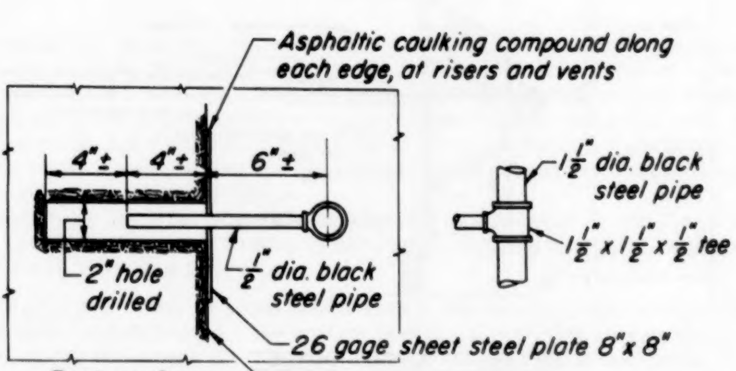
The Corps objects to the use of consolidation grouting as a general practice on the grounds that (1) it tends to destroy the natural drainage that exists in a foundation, and (2) generally it is a poor, if not questionable, substitute for deeper excavation.

Contact Grouting

Contact grouting is defined as the injection of a grout slurry at the contact of a structure with a vertical or nearly vertical rock surface for the purpose of sealing any water passages that may exist because of shrinkage. It is the Corps' practice to contact grout all high vertical or nearly vertical step-up



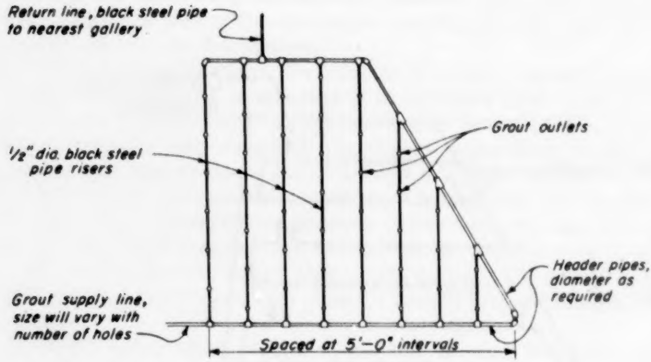
PIPING SYSTEM



DETAIL OF GROUTING OUTLET

CONTACT SURFACE GROUTING

FIGURE 1



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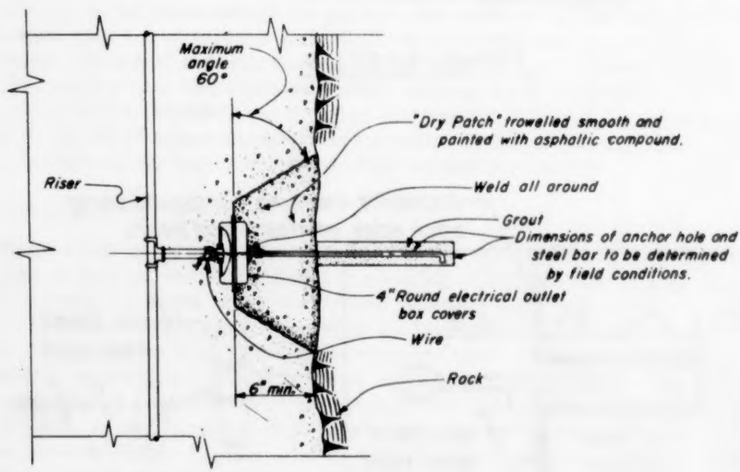


DETAIL OF GROUT OUTLET

CONTACT SURFACE GROUTING

NOT TO SCALE

FIGURE 2

faces in the abutments of concrete dams. After the structure is completed contact grouting is generally done through header-pipes leading from the gallery to the contact installation. Two contact installations have been used by the Corps and are shown in Figures 1 and 2. The installation shown in Fig. 1 is preferred. The installation is thoroughly washed before grouting. Water is circulated in the installation while any curtain grouting operation that might clog the installation is in progress.

Dental Treatment

Dental treatment is defined as the operation of cleaning and filling with mortar grout or concrete of cavities, localized pockets of weathered rock, potholes, faults or other foundation flaws that extend below the general foundation surface and that are too small or inaccessible to permit the use of standard excavation and concrete placement methods. Accessibility for cleaning and filling is gained by large diameter calyx drilling, shafting or trenching.

Slush Grouting

Slush grouting is defined as the filling with a sanded-grout or mortar of surface irregularities and open fractures in a rock foundation on which earth fill is to be placed. Its purpose is to provide a suitable surface on which to place the earth fill and to reduce seepage at the contact of the fill with the foundation.

Methods

The Corps' guide specifications for foundation grouting prescribe the stage grouting method. While a packer may be used to seal off a stage of a grout hole during the initial application of grout to permit the use of higher pressures, the conventional packer grouting method, wherein the grout holes are drilled to the full depth of the treatment and grouted through a packer at successively shallower depths and at progressively reduced pressures, has had a very limited use by the Corps.

Before outlining the stage grouting method as practiced by the Corps certain terms that are used to specify and control drilling and grouting procedures are defined as follows:

Section is a linear or areal subdivision of the overall distribution of the grout holes established for the control of drilling and grouting operations. An example would be a limited reach along the line of the grout curtain in which grout hole drilling and grout injection may not be carried on concurrently.

Zone is a subdivision of the overall depth of grouting treatment fixed by design requirements for the control of drilling and grouting operations.

Stage is a partial or complete depth of a grout hole within a zone. The depth of a stage depends upon the bedrock conditions encountered and is generally determined by a loss or gain of drill water.

Split spacing is the systematic reduction of the grout hole spacing by drilling holes midway between previously drilled holes. The first set of holes is

referred to as primary holes, the second and succeeding sets as secondary, tertiary, and so on holes, respectively.

Washing is the process of removing the drill cuttings and sludge from a grout hole by the application of jets of water or alternating jets of water and air at the bottom of the hole.

Pressure washing is the process of removing mud or other soft fillings from potential water passages within a foundation by the application of water and air under pressure to grout holes. It is, in effect, a sluicing operation in which alternate currents of water and air are introduced under pressure into a hole, and the soft fillings ejected through nearby surface vents which may be either open grout holes or open fractures.

Pressure testing is the process of testing the leakage characteristics of a grout hole by applying water to the hole under known pressure, and measuring the rate of inflow. Accomplished with the grouting plant immediately prior to grout injection, pressure testing serves the threefold purpose of (1) providing a means of testing the grout pump, grout delivery line and connections to the hole, (2) minimizing the problem of absorption of water from the grout by the foundation material and (3) providing an aid to the determination of grout mix design.

Stage Grouting

Stage grouting under the Corps' specifications involves the following procedures. The primary holes in a given section of the first zone are drilled, say on 20-foot centers, to their first stage of depth as defined above, grouted at low pressure, and the grout within the holes removed by jetting or other methods before it has set sufficiently to require redrilling. Before grout injection is started in any hole, the hole is washed, pressure tested, pressure washed if necessary, the nearest two holes in advance of the hole to be grouted are drilled, and the nearest hole is thoroughly washed. In the case of concrete dams, the grouting of any hole is delayed until all concrete within 100 feet has reached full height or a height that will permit the use of the maximum working pressure without endangering the structure.

After all first-stage grouting of the primary holes in a given section of the first zone has been completed, and a minimum period of 24 hours has elapsed since completion of grouting operations in any given hole, the holes are deepened to their second stage of depth and grouted at higher pressure. The process of cleaning, deepening, washing, pressure testing, pressure washing and grouting at higher and higher pressures is continued until the predetermined depth of the first zone is reached. If any stage of a hole is found to be adequately tight as determined by pressure testing, e.g. takes less than 1.0 cubic feet of water in 10 minutes, grout injection in that stage is omitted and the hole left open for drilling in the next lower stage.

Secondary holes, located midway between the primary holes, are then drilled and grouted in accordance with the procedures outlined for the primary holes. The process of reducing the grout-hole interval by the split-spacing method and of drilling and grouting by stages is continued until the ground in a given section is satisfactorily tight to the full depth of the first zone. Using the holes of the first zone, successively lower zones are drilled and grouted by the same procedures until the final depth of curtain or grouting treatment is reached. Other sections are drilled and grouted in like

manner until the overall treatment is satisfactorily accomplished. Ordinarily the grout curtain for moderately high or high concrete dams is divided into three zones of depth.

Generally, the grout in each stage is applied at the top of the holes in order that each stage of each hole finally may be subjected to the maximum allowable pressure. However, exceptions will arise, due to bedrock conditions, especially in grouting the abutments and cutoff trenches of earth dams, when it will be found advantageous to set a packer at the bottom of the first stage, and sometimes the second stage also, in order that higher pressures may be resorted to in the lower stages without lifting the foundation or causing troublesome breakouts of grout at the surface. In such cases the low pressure first stage, or first and second stages, are finally regrouted from the top of the hole at the maximum allowable pressure.

To facilitate pressure washing operations in fractured ground containing clay fillings, it has been found desirable occasionally to depart from the split spacing method of determining the drilling sequence and to drill the holes in groups on relatively close centers initially.

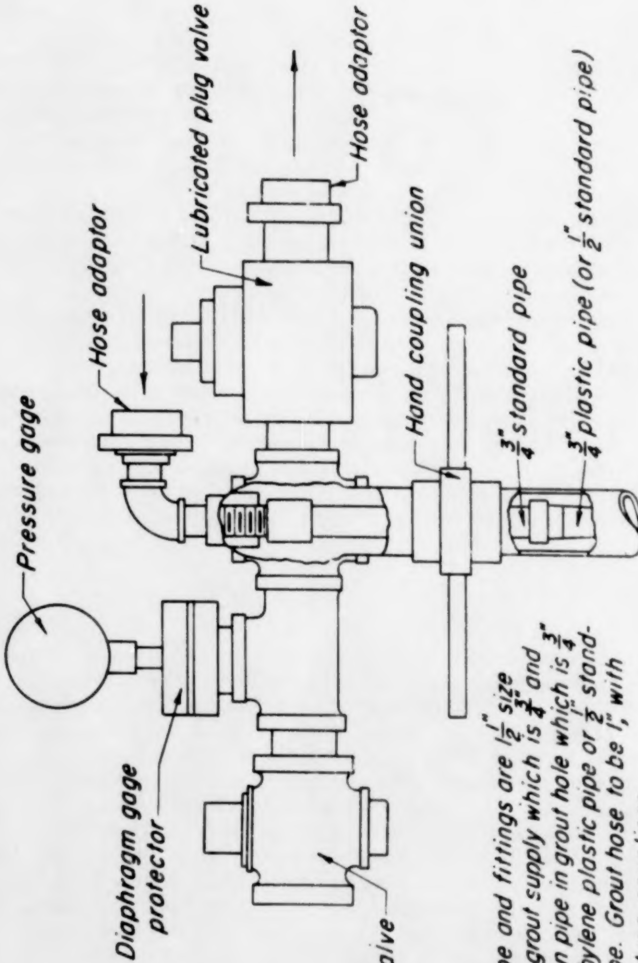
The Corps adopted the stage grouting method because of its overall effectiveness and the flexibility it provides for meeting localized conditions. Leaky zones are treated in the order in which they are encountered in the grout holes, thereby creating a progressively deeper barrier to the upward migration of the grout. Therefore, higher and higher pressures may be applied from the top of the grout holes to progressively deeper stages with a minimum of danger of upheaval. The principal disadvantages of the method are (1) the necessity for cleaning the holes of grout after each stage of grouting and (2) the interruption to drilling and grouting operations and the added cost that result from the necessity of moving the drill following each stage of drilling and grouting.

Circuit Grouting

Recently, at the Folsom Dam in California, the Corps expanded its specifications for foundation grouting to include provisions for circuit grouting in combination with the stage grouting method. Certain important advantages appeared inherent in the circuit grouting system of grout injection, and its investigation under field operating conditions was considered advisable.

In circuit grouting the supply pipe and return pipe are connected to the grout hole or grout nipple by means of a special header that permits the injection pipe to be extended to the bottom of the grout hole. Thus the grout slurry is made to circulate from the agitator to the bottom of the hole, thence up the annular space between the injection pipe and the wall of the grout hole to the header, thence through the return pipe from the header to the agitator. Pressure is controlled by a valve in the return pipe. Fig. 3 shows the circuit grouting header used with either standard or plastic pipe. Fig. 4 shows the circuit grouting header used with thin-wall flush-joint steel tubing.

Although standard pipe may be used to extend the grout line from the header to the bottom of the hole, it is objectionable because (1) the sleeve couplings greatly reduce the annular space through which the return grout flows and (2) the couplings obstruct the removal of the pipe in caving or ravelling holes. It was found preferable to use polyethylene plastic pipe or thin-wall flush-joint steel tubing. The choice between plastic pipe and steel tubing

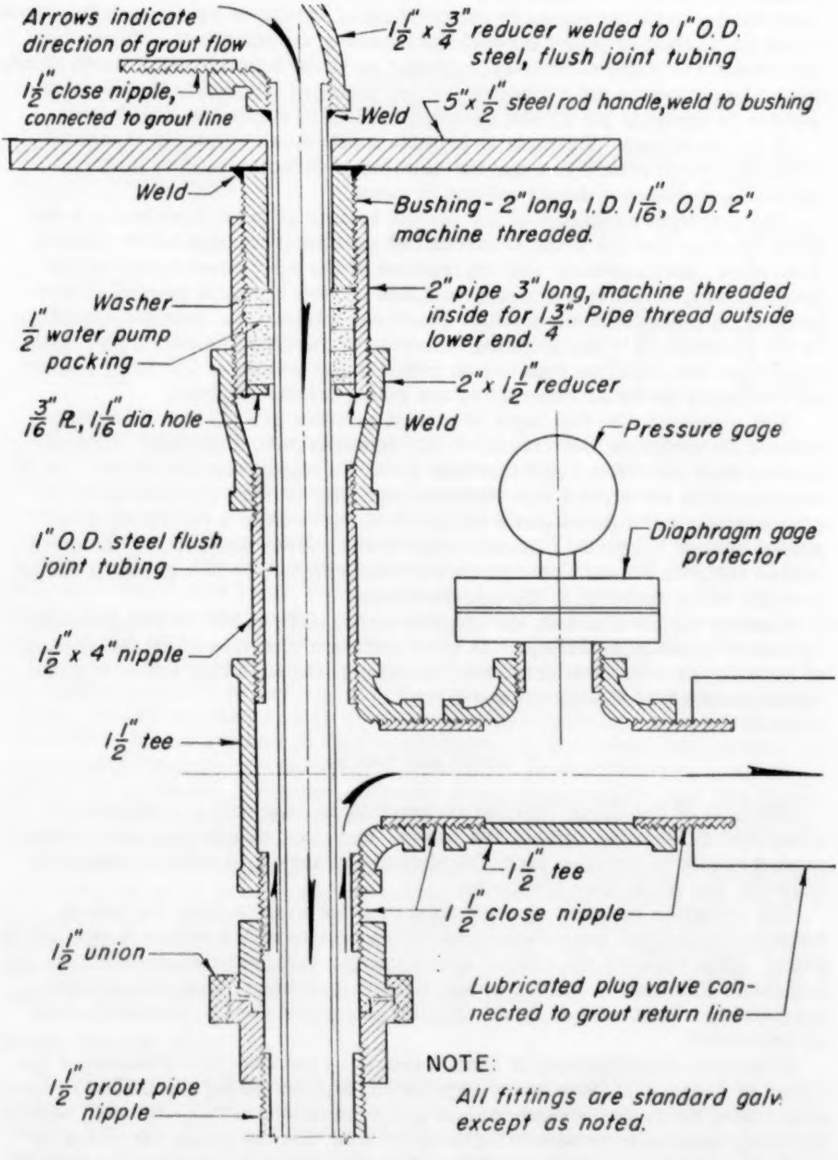


NOTE:

All pipe and fittings are $1\frac{1}{2}$" size except grout supply which is $\frac{3}{4}$" and injection pipe in grout hole which is $\frac{3}{4}$" polyethylene plastic pipe or $\frac{1}{2}$" standard pipe. Grout hose to be 1", with screw type couplings.

CIRCUIT GROUTING HEADER

FIGURE 3



CIRCUIT GROUT HEADER WITH STUFFING BOX

depends upon the conditions of the work. Plastic pipe is flexible and is particularly well suited to gallery grouting where headroom is limited. The steel tubing is better suited to the grouting of caving or ravelling holes since it can be washed or jetted through the debris more effectively. Since such conditions are more commonly encountered in the foundations of earth dams, where headroom is not a limiting factor, tubing is frequently the more desirable in applying the circuit grouting method to earth dam foundations.

It has been found desirable to provide a 100-mesh vibrating screen between the grout return line and the agitator in order to remove materials picked up by the circulating column of grout.

The principal advantage of the circuit system of grout injection results from the fact that the grout is circulated constantly throughout the system. Therefore, sedimentation and segregation of the solid constituents of the grout slurry, which is a primary problem of slow holes, is prevented and premature plugging of the smaller fractures minimized. Another advantage is its adaptability to the grouting of caving or ravelling holes. Under such conditions the injection pipe can be jetted to the bottom of the holes and the accretionary material removed by the rising column of grout.

The principal disadvantages of circuit grouting are (1) that it requires more time to install and to remove the injection pipe, especially if jointed pipe is used and (2) in holes that take grout so rapidly that no return flow is maintained in the circuit, the injection pipe may become grouted in by the stagnant column of grout above the point of injection in a relatively short period of time. A further disadvantage results from the fact that the grout holes cannot be closed under pressure after completion of a grouting operation due to the removal of the injection pipe.

Used in conjunction with the stage grouting method, the circuit grouting system of grout injection appears to be the most effective of all the systems of injection provided the grout hole conditions are such that a flow of grout can be maintained throughout the circuit.

Grout Mix Design

The bulk of the Corps' foundation grouting is done with a grout slurry composed of Portland cement and water. However, extensive use is made of sanded grouts in grouting large foundation voids such as solution channels, cavities, and large open fractures.

The principal problem of using sand in grout results from its strong tendency to undergo segregation and sedimentation which makes it difficult to pump. Until recently the Corps' specifications required the use of flyash and a suitable fluidifier in sanded grouts to inhibit sedimentation and promote pumpability. Bentonite or other claylike materials were prohibited for use as fluidifiers.

Extensive investigations of sanded grouts by the Concrete Division of the Corps of Engineers' Waterways Experiment Station during the last two years have shown first, that a sand-cement grout containing a well-rounded, natural, silicious sand having a specific gravity of 2.62, and the gradation shown in Table 1, can be pumped successfully when the sand-cement ratio does not exceed 2.0, second, that a grout having a substantially higher sand-cement ratio can be made pumpable by the addition of a finely divided, silicious mineral filler such as flyash, diatomite, pumicite or loess, and third that a sanded

TABLE 1

SAND GRADATION

FOR SANDED GROUT PUMPING TESTS

SIEVE DESIGNATION (U.S. STD. SQUARE MESH)	CUMULATIVE PERCENTAGE BY WEIGHT	
	Passing	Retained
8	100	0
16	95 - 100	0 - 5
30	60 - 85	15 - 40
50	20 - 50	50 - 80
100	10 - 30	70 - 90
200	0 - 5	95 - 100

grout containing a large percentage of sand can be made pumpable by the addition of relatively small amounts of bentonite. Similar results have been obtained with sands manufactured from limestone and trap rock. Table 2 summarizes the test data obtained with natural sands. Table 3 summarizes the results obtained with manufactured sands. The part by weight of sand shown in these tables is the maximum that could be pumped through 200 feet of 3/4-inch hose coiled on a 5-foot radius after a 15-minute shutdown. The hose, immediately after leaving the pump, rose 13 feet, passed over a water pipe on a radius of 5 inches, returned to the pump level, then entered the 5-foot radius coil. Mixing was done in a conventional paddle mixer. All batches had a consistency of 125 to 150 degrees of torque as measured by a piano wire torque consistency meter, with consistency also determined with the Stormer viscosimeter. As a result of these investigations, the Corps now permits the use of sanded grouts containing a suitable finely divided silicious mineral filler such as flyash, diatomite, pumicite or loess without the use of a fluidifier. In special cases, where strength is not an important consideration, the use of bentonite is also permitted.

The required strength of a sanded grout will vary according to the purpose for which the grout is used and the quality of the foundation into which it is injected. While a sanded grout of comparatively high strength might be required in grouting large voids in a dam foundation, one of comparatively low strength would be satisfactory for sealing a cavity in a reservoir rim or in the contact grouting of a tunnel lining in soft rock.

Water-Cement Ratio

Field experience and laboratory research have led the Corps to believe that grouts of high water-cement ratio are unnecessary and undesirable, primarily because of the poor quality of the end product but also for economic reasons. Therefore, the maximum water-cement ratio of neat cement grout permitted under the Corps' specifications is 3 or 4 and the bulk of the Corps' foundation grouting is done at ratios of 1 or less.

It is the practice of the Corps to express the water-cement ratio in terms of cubic feet of water per bag of cement.

TABLE 2
SANDED GROUT PUMPABILITY TEST DATA

PARTS BY WEIGHT				COMPRESSIVE STRENGTHS psi	
NATURAL SAND	CEMENT	FLYASH	WATER	7 DAYS	28 DAYS
3.10	1.0	0.11	0.88	1195	2210
3.80	1.0	0.25	1.07	960	1720
4.30	1.0	0.43	1.16	800	1270
5.40	1.0	0.67	1.35	570	855
6.50	1.0	1.00	1.62	435	715
		PUMICITE			
3.10	1.0	0.11	0.94	1095	1965
3.80	1.0	0.25	1.11	715	1260
4.30	1.0	0.43	1.27	600	1080
5.40	1.0	0.67	1.67	275	575
6.50	1.0	1.00	2.04	170	380
		LOESS			
2.80	1.0	0.11	0.90	1300	2355
3.40	1.0	0.25	1.04	845	1610
4.30	1.0	0.43	1.34	400	765
5.00	1.0	0.67	1.52	385	665
6.00	1.0	1.00	1.88	255	410
		DIATOMITE			
3.00	1.0	0.03	0.95	860	1840
3.25	1.0	0.06	1.00	745	1900
3.90	1.0	0.11	1.19	585	1035
5.00	1.0	0.25	1.54	235	495
5.70	1.0	0.43	1.87	150	485
7.10	1.0	0.67	2.37	120	410
9.00	1.0	1.00	3.18	55	235
		BENTONITE			
4.00	1.0	0.025	1.35	375	670
6.00	1.0	0.050	2.03	175	308
11.00	1.0	0.10	3.25	43	63
14.00	1.0	0.20	5.88	15	25
24.00	1.0	0.30	9.76	10	10
32.00	1.0	0.40	12.17	--	--

NOTE: See Table 1 for sand gradation

TABLE 3

SANDED GROUT PUMPABILITY TEST DATA

PARTS BY WEIGHT				COMPRESSIVE STRENGTHS psi	
LIMESTONE SAND	CEMENT	FLYASH	WATER	7 DAYS	28 DAYS
1.90	1.0	0.11	0.73	1650	3370
2.50	1.0	0.25	0.80	1200	2590
3.20	1.0	0.43	1.08	880	2395
3.80	1.0	0.67	1.21	795	1990
5.00	1.0	1.00	1.58	595	1385
TRAP					
1.94	1.0	0.11	0.73	1435	3420
2.50	1.0	0.25	0.90	1230	2405
2.90	1.0	0.43	1.03	1065	2190
3.80	1.0	0.67	1.24	795	1770
4.50	1.0	1.00	1.49	710	1450
LIMESTONE		LOESS			
1.90	1.0	0.11	0.74	1595	3000
2.50	1.0	0.25	0.92	985	1545
2.90	1.0	0.43	1.05	735	1350
3.80	1.0	0.67	1.35	445	845
4.50	1.0	1.00	1.62	265	535

NOTE: Sand gradation same as shown in Table 1 except percentage passing 100 sieve = 0

Grouting Pressures

A rigid set of rules for the control of grouting pressures is impracticable because of the many variable factors that affect the problem. Some of the factors to be considered are the hydrostatic pressure to be imposed on a dam foundation by the reservoir, the weight of the structure at the time of grouting, the weight of the foundation material above the zone of grout injection, the type and attitude of the foundation rock, the consistency of the grout, the degree of tightness of the grout hole, and the extent to which the rock above the zone of grout injection has been sealed previously by grouting operations.

In grouting the foundations of gravity-type concrete dams from a gallery, it is a general practice of the Corps to use a maximum working pressure at least twice the hydrostatic pressure to be imposed on the foundation by the reservoir. Since the weight of concrete is approximately 2-1/2 times that of water, such pressures are attainable with safety in any type of foundation if grouting is delayed until the structure reaches full height.

In grouting from the surface of a foundation, when no load has been superimposed, the problem of uplift becomes a major one, especially when grouting flat-lying sedimentary rocks. The desirable maximum working pressure frequently cannot be attained with safety. However, it may be approached by a careful application of the stage grouting method. The rule of thumb that the pressure in pounds per square inch at the point of injection should not exceed the depth in feet to the point of injection is perhaps the one most commonly applied in grouting foundations under such conditions. However, this rule does not take into account the physical conditions of the foundation rock, the consistency of the grout, the degree of tightness of the grout hole, or the

progressive reduction in the area upon which the pressure may act that takes place as grouting proceeds, especially when the stage grouting method is employed.

It is the Corps' practice to specify the maximum allowable working pressure that shall be used on a grouting job, and to leave the determination of pressures for specific cases of grout hole spacing, stage and zone to the field supervisors. Fig. 5 represents a diagram which is used as a rough guide in determining the safe grouting pressure for various conditions of foundation materials.

Grout Hole Drilling

While the Corps' guide specifications do not prohibit the use of percussion drills in drilling shallow grout holes, provided the foundation rock is of a type that will produce granular cuttings, rather than slimes, its standard practice is to use rotary drills equipped with either diamond plug bits or diamond coring bits. Cores are not required from the grout holes but the drilling of a limited footage of NX core borings is provided for to explore the results of the grouting program. The Corps also makes extensive use of its borehole camera in exploring the results of grouting operations.

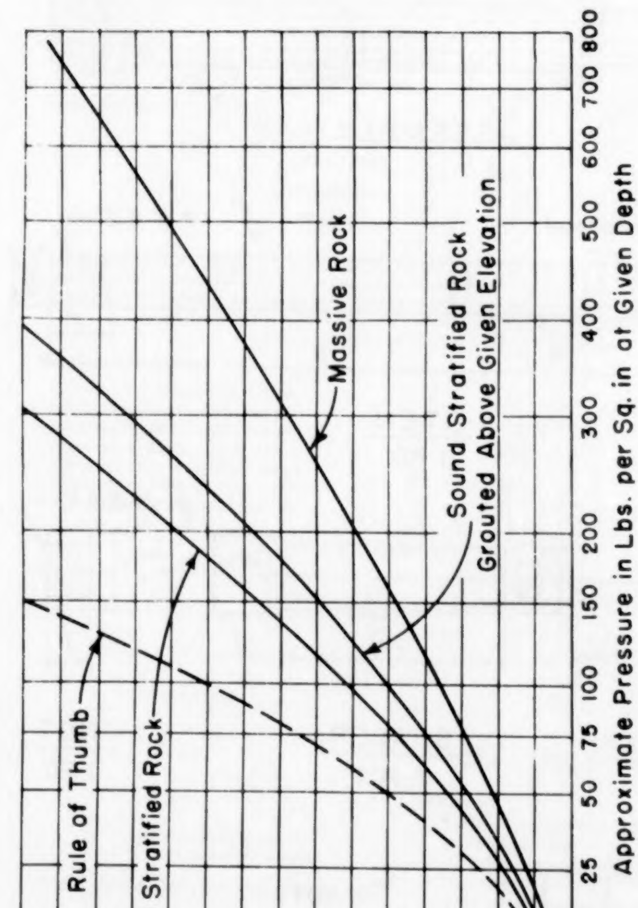
For a number of years the Corps specified the use of diamond core drills of NX size (hole diameter 3 inches) for drilling grout holes. This preference was based on the assumptions that hole size was a factor in grout injection, and that the NX hole was the minimum size in which the conventional mechanically expanded packer could be used effectively. Later experience indicated that the diameter of a grout hole generally had little effect on the amount of grout that could be injected and that packers of special design could be used effectively in holes as small as size EX (diameter approximately 1-1/2 inches). Therefore, the EX bit has become the standard for drilling grout holes. Not only does this size effect important economies, but also it provides a grout hole of the same diameter as that of the grout supply and grout return lines use by the Corps, which appears desirable. The packer used in EX holes is shown in Fig. 6.

Curtain grout holes, when drilled from a gallery, are inclined upstream 10 to 15 degrees from the vertical and may also be given an inclination in a direction that will effectively intersect the most prominent rock jointing, if such jointing is essentially vertical. The purpose of the upstream inclination is to provide divergence between the grout holes and the foundation drain holes. Fig. 7 shows the relationship between the grout holes and the drainage system. For many years a 5 by 7 foot gallery was the standard size. To provide more head-room for drilling operations, a 5 by 8 foot gallery is now preferred.

Grouting Equipment

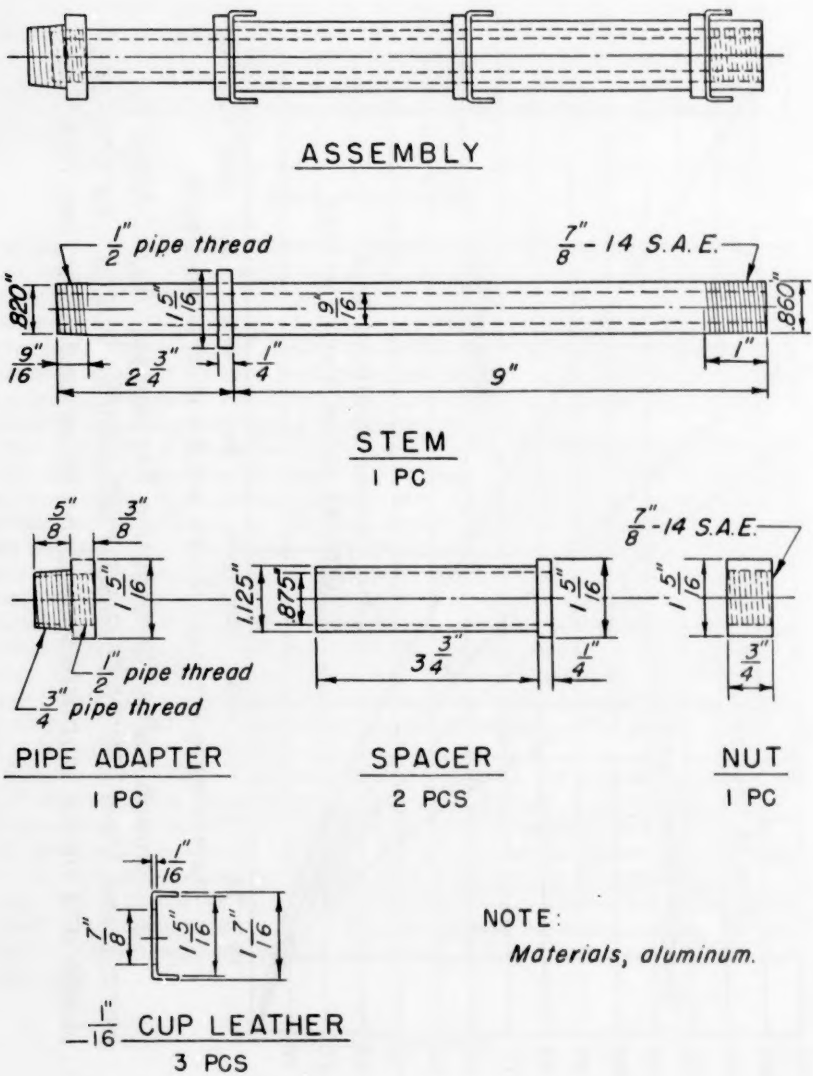
The equipment used by the Corps for mixing and injecting both neat cement grout and sanded grout is similar to that which has been more or less the standard of the construction industry in the United States for many years. Fig. 8 is a schematic drawing of the grout mixing and pumping plant.

The essential features of this plant are, first, an air motor-driven paddle mixer with hopper feed and water meter, second, two grout sump tanks



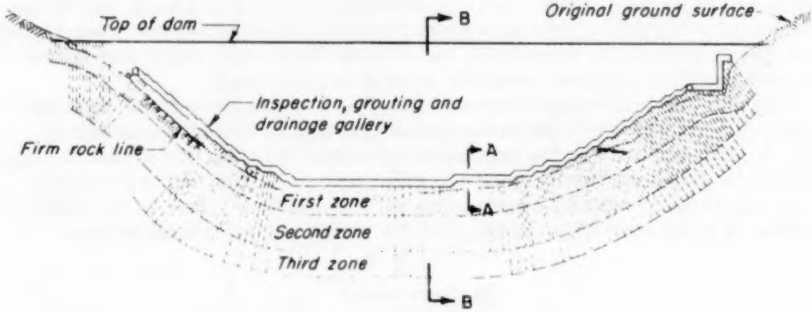
Rough Guide for Grouting Pressures. (Reproduced by permission from "ENGINEERING FOR DAMS - VOL. I" by Creager, Justin and Hinds, published by John Wiley and Sons, Inc. 1945.)

FIGURE 5



EX GROUT PACKER

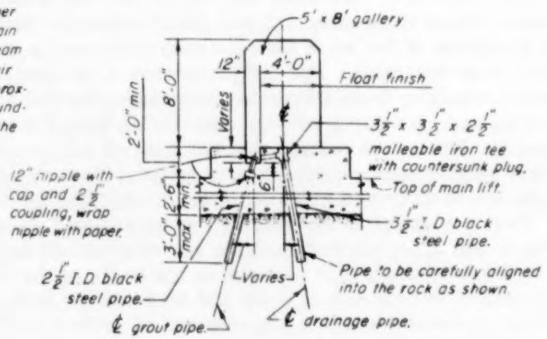
FIGURE 6



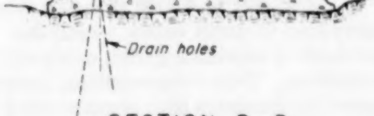
TYPICAL PROFILE OF CURTAIN GROUT HOLES
(ALL GROUT HOLES NOT SHOWN)

NOTES:

Grouting may require any number of stages per zone. Grout curtain holes should be inclined upstream as shown in Section A-A. Their lateral deviation, if any, from approximate normality with zone boundary may be determined from the geology at the site.



SECTION A-A



SECTION B-B

NOTES:

Gallery floor should be sloped toward gutter at approx. ⅛" per foot.
All fittings except plugs, should be made up tight.
Plug all pipes during construction, plugs to be greased with waterproof grease.
During concrete placing, great care should be taken to prevent deflecting of embedded pipes.
Metal water seals not shown.
Drain holes, 3" dia., 15' max. spacing, with pipes inserted in between as required for drilling intermediate holes. Drain holes drilled from gallery after grouting.
Grout holes, 1½" min. dia., spaced as required inclined upstream and grouted with high pressure from gallery after sufficient concrete cover has been placed.

CONCRETE DAMS GROUTING AND DRAINAGE DRILLING

FIGURE 7

equipped with air motor-driven paddle agitators, third, two air-driven double-acting, publex slush pumps of the long stroke type, having large valve openings, removable valve seats and removable liquid end liners, one of the pumps being a standby, fourth, a grout header for connecting the grout supply and return lines to the grout hole, and fifth the necessary gages and valves properly located to insure accurate control of pressures.

The conventional header shown in Fig. 8 will be replaced in the next revision of the Corps' manual on foundation grouting by the header shown in Fig. 9. This header has the important advantage of having the pressure gage located in a position that permits the grout hole to be shut in for pressure drop observations without interrupting the circulation of grout in the plant system, a procedure which is not possible with the conventional header.

Payment Items

No other type of dam construction work has a larger element of uncertainty with respect to quantity requirements than foundation grouting. Therefore, the Corps reserves the right in its specifications to increase by as much as 100 per cent or to eliminate any part of the entire drilling and grouting progress without increasing unit bid prices, should conditions disclosed during the progress of the work indicate such increase or decrease in the amount of work to be desirable. For the protection of the contractor against possible losses resulting from important decreases, the bid schedule for foundation drilling and grouting generally contains an item for mobilization and demobilization. When it is certain that the drilling and grouting quantities will be large, an alternative methods may be used wherein each of the major bid items for drilling and grouting is subdivided and the bid quantity in each of the first sub-items is represented as an assured minimum to which the contractor can apply his mobilization and demobilization costs.

Grout hole drilling is paid for on the basis of the linear feet of holes actually drilled in concrete or rock and includes the cost of washing, pressure testing, pressure washing and removal of grout from the holes for deeper stage drilling. Should the contractor be directed to leave the grout in a hole until it takes a hard set to control back-pressure conditions, thus necessitating a redrilling operation, he is paid therefor at one-half his bid price for grout hole drilling.

Cement, sand, and mineral filler used in grouting are paid for on the basis of the number of cubic feet of each injected into the grout holes. Grout injection is paid for on the basis of the number of cubic feet of solids injected into the holes. All connections of the grout supply line to grout holes, which the project engineer finds necessary for the purpose of injecting grout, are paid for at a fixed rate of $5 to $10 for each connection. This compensation, based on an estimate of cost, is intended to mitigate the hardship that results when the number of connections per hole becomes excessive.

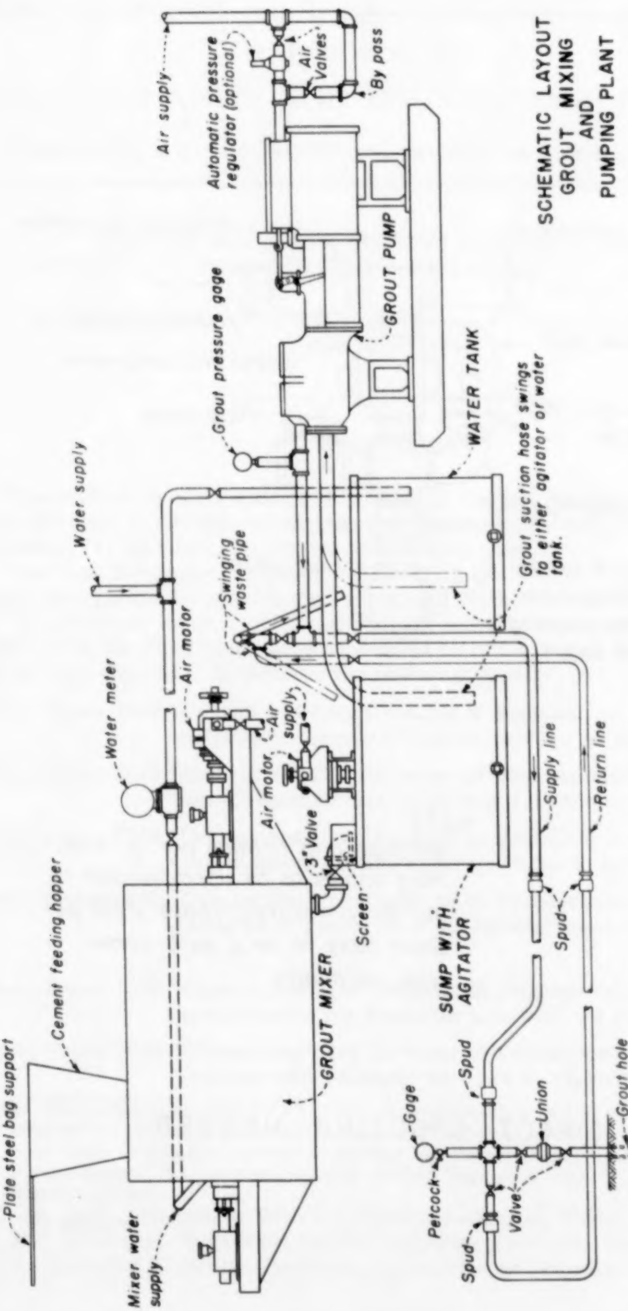
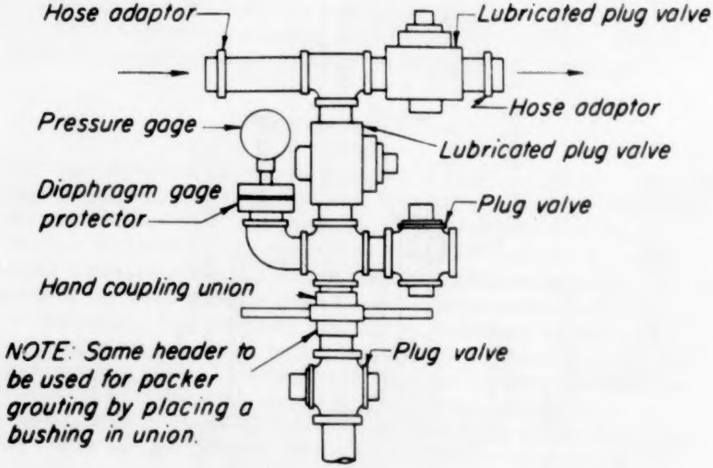

SCHEMATIC LAYOUT
GROUT MIXING
AND
PUMPING PLANT
Air supply
Automatic pressure regulator (optional)
Air Valves
By pass
GROUT PUMP
Grout pressure gage
Water supply
WATER TANK
Grout suction hose swings to either agitator or water tank.
Swinging waste pipe
Air motor
Water meter
Air supply
Air motor
3" Valve
Screen
Supply line
Return line
Spud
SUMP WITH AGITATOR
Cement feeding hopper
Plate steel bag support
GROUT MIXER
Mixer water supply
Gage
Spud
Union
Grout hole
Petcock
Valve
Spud
GROUT HEADER

FIGURE 8



NOTE:
All pipe and fittings are $1\frac{1}{2}''$ size.
Plug valves to be used throughout for pressures above 250 psi.
Grout hose to be 1" with screw type couplings.

DIRECT GROUTING HEADER

FIGURE 9

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

CEMENT AND CLAY GROUTING OF FOUNDATIONS: EXPERIENCE OF TVA WITH CLAY-CEMENT AND RELATED GROUTS

George K. Leonard,[1] M. ASCE and Leland F. Grant,[2] A.M. ASCE
(Proc. Paper 1552)

FOREWORD[a]

The work of the Committee on Grouting of the Soil Mechanics and Foundations Division is divided among four task committees, concerned with (1) bituminous, (2) chemical, (3) cement and (4) clay grouting.

The Task Committee on Chemical Grouting has completed its task and its report was published in the Division Journal, November 1957.

"The Symposium on Cement and Clay Grouting of Foundations" is a "joint venture" of the Task Committee on Cement Grouting and of the Task Committee on Clay Grouting. It includes the following papers:

Proc. Paper 1544 "Cement and Clay Grouting of Foundations: Present Status of Pressure Grouting Foundations" by A. Warren Simonds

Proc. Paper 1545 "Cement and Clay Grouting of Foundations: Grouting with Clay-Cement Grouts by Stanley J. Johnson

Proc. Paper 1546 "Cement and Clay Grouting of Foundations: The Use of Clay in Pressure Grouting" by Glebe A. Kravetz

Proc. Paper 1547 "Cement and Clay Grouting of Foundations: The Use of Admixtures in Cement Grouts" by Alexander Klein and Milos Polivka

Proc. Paper 1548 "Cement and Clay Grouting of Foundations: Suggested Specifications for Pressure Grouting" by Judson P. Elston

Proc. Paper 1549 "Cement and Clay Grouting of Foundations: Pressure Grouting with Packers" by Fred H. Lippold

Note: Discussion open until July 1, 1958. A postponement of this closing date can be obtained by writing to the ASCE Manager of Technical Publications. Paper 1552 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SM 1, February, 1958.

1. Chf. Engr., Tennessee Valley Authority, Knoxville, Tenn.
2. Eng. Geologist, Tennessee Valley Authority, Knoxville, Tenn.
a. By Raymond E. Davis, Chairman, Committee on Grouting

Proc. Paper 1550 "Cement and Clay Grouting of Foundations: French Grouting Practice" by Armand Mayer

Proc. Paper 1551 "Cement and Clay Grouting of Foundations: Practice of the Corps of Engineers" by Edward P. Burwell, Jr.

Proc. Paper 1552 "Cement and Clay Grouting of Foundations: Experience of TVA with Clay-Cement and Related Grouts" by George K. Leonard and Leland M. Grant

These papers are based largely on the results of successful grouting operations for a wide variety of foundation conditions but also are based on the results of laboratory investigations and to some extent on the ideas and opinions of designing engineers who are concerned with foundation problems.

The information contained in the papers has been assembled for the particular benefit of those responsible for the foundation treatment of structures. Recognizing that these papers by no means represent the "last word" in good practice and that there may be many others concerned with foundations who could contribute to our knowledge of cement and clay grouting, discussion is earnestly sought.

SYNOPSIS

Most of the dams built by the Tennessee Valley Authority have required large amounts of grouting for foundation preparation. Clay-cement grout has been used where feasible, and thereby, it has been possible to obtain safe and watertight foundations at a much lower cost than by using usual neat-cement grout.

INTRODUCTION

The Tennessee Valley Authority has built 20 dams since the beginning of construction in 1933. The Tennessee River Valley is an area where foundation grouting, even under low head dams, is a must. This can readily be understood from the rock types present in the area and by a study of its physiographic history.

The geology of the TVA region will not be described in this paper in great technical detail, although a comprehensive grouting plan depends chiefly on a thorough knowledge of what geologic conditions are involved. Except for the mountainous area on the east side of the Basin, the foundation work in the Tennessee Valley is in shale, limestone, dolomite and sandstone; and, of these, the joints, bedding planes and solution channels in the limestone have offered the greatest problem. A more detailed description of subsurface conditions at the several TVA dams is given in Technical Report No. 22. [1]

Admittedly, good geologic data is indispensable during the search for a dam site, or later during the more extensive exploration during the design stage prior to construction; but, during actual construction need of this important item is sometimes great. The original exploration holes are, of course, carefully logged by the geologist and so, too, are the subsequently

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1. Geology and Foundation Treatment, Tennessee Valley Authority, Technical Report No. 22, 1949.

drilled grout holes during construction. From the latter and from exposures in excavations a great volume of additional information is obtained on even the minutest details of geology. This permits the plotting of geologic sections which are used constantly by the construction staff. It is imperative that the construction engineer have the most exact and detailed information it is possible to obtain. It is possible to make many of the early decisions on general knowledge or experience, but, during construction, an intimate knowledge of every stratum, seam, cavity and geologic structure, however minor, is not only desirable but, in most cases, necessary. These detailed drawings and reports of the geologist are some of the tools used by the engineer in planning and laying out a comprehensive foundation treatment program.

Selection of Grouting Materials

In the treatment of the foundations of the TVA dams, three important factors were considered in the selection of the materials used in the grouting. The first and foremost was the type of structure. Second, and still of prime inportance, was local subsurface conditions. The third factor, the materials available, was always considered in the planning of grouting work. The use of large volumes of untreated local clays in the grout mixes made this third factor very important.

TVA's experience with clay-cement grout in foundation treatment seems to be overshadowed by the neat-cement grouting it has done and the extensive foundations it has literally manufactured in spite of the treacherous, unpredictable, subterranean solution channels and caverns in the highly soluble limestone that underlies most of the major dams of the Tennessee River watershed. It would be exaggerating a little too much to say that TVA's seven dams (actually nine, but it did not build two of them) on the Tennessee River were located first and the foundations explored and built later. But local conditions, including requirements for navigation, governed their location to a great extent and the foundations were taken as they were found.

TVA has never used clay-cement grout in the river channels for grout cutoff curtains, preferring the more durable and reliable, although more expensive, neat-cement mixes. TVA believes it unwise to try to make a permanent clay-cement cutoff grout curtain of pit-run clay in either thin-bedded, jointed rocks or in large solution channels in foundation rock which are subject to direct headwater pressure or where high compression loads must be carried. The risk of piping or foundation failure is too great to take a chance on the use of a relatively soft material such as clay even when strengthened by the addition of cement. In some cases, piping would cause loss of water, and under earth dams might even cause failure of the dam. Under concrete dams, build-up of uplift pressure downstream from the cutoff curtain might cause failure of the structure.

Use of Bentonite Clay

Bentonite has been used on several TVA dams. It has frequently been used under the large cofferdams which were located on limestones and dolomites. In most cases it was mixed with neat-cement and water, but whenever large and excessive takes of grout were involved, clay, sand, sawdust and rock flour have been added. This was nicknamed "prescription grout." When it

was used, in most cases, it was selected for its swelling and jelling properties. At Kentucky Dam "prescription grout" was used to fill voids below the ends of 75-foot long steel sheet piles driven in the cofferdam cells where they seated on the irregular rock foundation. In the cofferdam grouting, the most readily available material was a fine-grained white sand from the Ripley formation. The clay and silt sizes which usually make such natural deposits usable were deficient and the bentonite was used to make the grout more workable. Bentonite was also used at Guntersville, Chickamauga, Fort Loudoun, Cherokee and Douglas dams.

Use of Local Alluvial Clays

Although in the Tennessee River area geologic conditions limit the use of clay-cement grout, it has been used extensively in situations where its use was found to be economical and justifiable. Its use has been approved along reservoir rims, especially when the side slopes leading up to the rim are flat and seepage, if any, will have negligible velocities; for filling cavities and sink holes under earth fill slopes and switchyard fills where subsidence of the natural ground into the sink might occur, causing simultaneous subsidence of the fill; for cutoff curtains under saddle dams in the rim of the reservoir; and for tightening up fills and in permeable conditions under cofferdams.

The limestones and dolomites that are massive and thick-bedded usually are the ones which contain the large solution cavities that can be grouted with clay-cement grout. It has been established with a high degree of certainty that the solution of the limestone and dolomite rocks of the Tennessee Valley has been controlled by, and aligned along, structural features in the rocks, such as bedding planes, joints, and faults. These, naturally, expose larger uninterrupted areas of susceptible rocks to the dissolving action of ground water in thick-bedded rocks than they do in the thin-bedded rocks. It is to be expected that an extensive jointing or fracturing system in a pure massive limestone rock would develop a group of very large cavities, whereas similar fracturing in a thin-bedded limestone would cause the development of smaller cavities.

Large cavities occurring at some sites are particularly well suited for grouting with clay-cement mixes since the large size makes it desirable to fill them with as cheap a material as possible, and the harsh and gritty mixes with grain sizes too large for fine joints can be injected rather easily into them. The original cave clay which occurs in these large cavities is difficult to remove. It can, however, be left in place whenever clay-cement grout can be used to plug the open part of the cavities. In many instances it has been noted that the original cave clay is compacted and consolidated by the injected grout. It has been observed as an interesting coincidence that at many dam sites a high percentage of the large open cavities which are suitable for this type of grouting are located in higher parts of the abutments or in distant rims and saddles where load bearing properties are not too important. In almost every instance where this material has been used, large and more or less open solution cavities in limestones and dolomites have been involved.

A summary shows that a total of 5,294,453 cubic feet of clay-cement grout was used at eight dams as follows:

Project	Quantity	Where Used
Guntersville Dam	29,475 cu. ft.	Consolidation grouting under earth embankment dams
Chickamauga Dam	720,833 cu. ft.	Consolidation grouting of earth fills and curtain grouting in reservoir rim
Fort Loudoun Dam	749,410 cu. ft.	Curtain grouting in north rim and saddle dam, and consolidation under switchyard
Cherokee Dam	297,652 cu. ft.	Curtain grouting in north and south reservoir rims
Douglas Dam	145,919 cu. ft.	Curtain grouting in north reservoir rim
Boone Dam	993,696 cu. ft.	For consolidation and curtain under earth fill, consolidation under switchyard, and for curtain grouting under right rim
South Holston Dam	1,711,171 cu. ft.	Curtain grouting for saddle dam
Fort Patrick Henry	646,297 cu. ft.	Curtain grouting in right rim

Equipment

Equipment for clay-cement grout is basically the same as that used for neat-cement grouting. It is housed under wood-frame and canvas sheds to protect all material and workmen allowing all-weather production. (figure 1) Clay is delivered to the storage pile under the mixer shed in scrapers. One-half yard mixers, batched and charged by hand, premix clay and water before discharging into rotary screens. The screened slurry is stored in agitator tanks and piped by gravity as needed into the grout mixer where the cement is added. The grout is pumped directly from the mixers to the hole being treated. The grout mixers and pumps are installed in duplicate so that production can be maintained while cleaning one set of equipment. It was found that where grout takes were extremely large, return lines between the header and the pump were not needed. In most cases, the required pressures could be maintained by wasting a small amount of grout at the header. The header installation is typical although a valuable innovation was the installation of a rubber diaphragm between the header and the pressure gauge. This helps to prevent the fouling of pressure gauges, especially in cold weather. As a general rule, it was found that it was well worth the extra expense to use the larger 2-inch instead of 1-1/4- or 1-1/2-inch hose and pipe which easily becomes plugged. The packer shown in figure 2 was especially job designed and built and it was found to be the most satisfactory. It had the advantage of clear openings throughout its whole length and did not clog with hair-like roots which could not be screened from the clay.

Mix Design

To determine the clay-cement ratio for the strongest and most impermeable grout, mix designs are of considerable importance. The use of large quantities of natural materials with only a minimum amount of preparation makes it difficult to maintain a uniform product. The mix designs are made in the soils laboratory from borrow pit samples which are tested for workability and mechanical analysis. Cylinders are then prepared and tested for permeability and compressive strength.

Field control of the mixes is based on the density of the water-clay slurry. Once the unit weight of a 40 percent mix of clay and water was established by experiment, the consistency of the grout could be controlled by varying the amount of slurry that was added to two bags of cement for each batch. In order to check the density of the slurry the inspector needed only to weigh 500 cc of the fluid and then check its weight on a calibrated graph. If a grout thinner than the normal was desired, clear water was added at the mixer which did not disturb the clay-cement ratio.

At South Holston Saddle Dam many difficulties were encountered in designing an economical clay-cement mix to suit the large quantities required for this work, that would provide an effective cutoff curtain and stay within limits of available equipment. Tests were made both in the field and laboratory to arrive at a workable mix. The clay was largely limestone residual deposits and varied somewhat from the typical alluvial clay normally used.

Several factors caused a variation in the consistency of the mix, chief of which was the wide variation in the unit weight of the clay because of the variable moisture content. The average weight of one cubic foot of loose clay with a moisture content of 30 percent was 62 pounds, or 43 pounds of dry clay and 19 pounds of water. After considerable experimentation, a mix consisting of 4.7 cubic feet water, 1 cubic foot cement, and 6 cubic feet of clay with a theoretical yield of 7 cubic feet of grout was decided upon. This mix by weight contained 94 pounds of cement, 258 pounds of clay, and established a ratio of 1 part cement to 2.7 parts of dry clay be weight.

The desired cement-clay ratio was maintained by ascertaining the percentage of clay solids in a given amount of clay-water solution which came from the mixers. Examples of the graphs prepared for the field control of the mix are shown in figures 3 and 4. By a series of density tests it was found that 15.2 cubic feet of clay-water solution containing 40 percent solids would yield the proper amount of solution for a two-bag batch. Each agitator tank was calibrated for this amount. Any deviation from the 40 percent solids was compensated for by adding or reducing the amount of clay-water solution introduced into the agitator.

When a more fluid mixture was desired, water was added at the agitator after the proper amount of clay solution had been introduced. This addition of water, of course, had no effect on the cement-clay ratio.

The clay-cement mixes which were designed for the grouting of large cavities were somewhat slower in setting than the neat-cement grout. In order to prevent the slow-setting grout from spreading into cavities outside of the area being treated, it was, at times, necessary to use accelerators. A great amount of judgment was needed to pick the times to use quick-setting grouts. If it was known that large cavities existed under a structure in an area being grouted it was always best to use lean mixes and continue to inject the grout intermittently until the cavities were filled. In places where it was

obvious from the records of the borings that the cavities were small and the consumption of grout in an area began to exceed the volume of the cavities under the structure, it was best to make the mix richer and use calcium chloride. It was found that in using fast-setting grouts the 1 to 1 and 1 to 2 clay-cement ratios gave the best results. Table 1 is an example of data supplied the grouting crews in the field in mixing usable fast-setting grouts. The initial setting time shown was determined by the standard method using a Vicat needle. It was, in all cases, firm enough to begin to pile up in a cavity but these tables show hardness well beyond the consistency for pumping.

The richest mixes have been in the ratio of one part cement to two parts clay. This was used where rather strong grouts were desirable or where the clay material tended to make a porous product and an impermeable one was desirable. The leanest grouts, which were used in places where excessive takes were involved, had a ratio of one part cement to twelve parts clay. This was found to be sound and watertight enough in the case of most grouting in large cavities in consolidation work.

Properties of the Clay

The clay-cement admixtures used by TVA in the grouting work discussed in this paper were prepared from local deposits. In all cases the borrow pits were adjacent to the structure being treated. The large volumes needed were excavated and hauled from the pits by the most economical means, usually in scrapers. It was screened to remove large lumps and rocks, sometimes before mixing with water and sometimes after, but always before the cement was added. This was all of the processing needed for the types of clay used.

The borrow pits which yielded the most workable and satisfactory clay were all deposits of alluvium occurring as old river terraces. The material is very common in occurrence and pits could usually be located conveniently to the work. The clay itself was a dark red, silty clay loam containing some colloidal iron. The typical mechanical analysis of the screened pit run material showed 25% clay, 45% silt and 30% sand. A typical chemical analysis is given in the following table.

	Percent
Silica	62.83
Alumina	18.26
Iron oxide	7.50
Magnesia	0.28
Phosphorous pentoxide	.18
Titanium dioxide	1.45
Manganous oxide	0.028
Potassium oxide	.61
Sodium oxide	.02
Loss on ignition (110° C-100° C)	7.16

Samples taken at the grouting units, not compressed and cured for 7 days in the fog room, developed an average shearing strength of 48 pounds per square inch. The average linear shrinkage amounted to one percent of the total height of the samples. Tests of the raw clay as used showed moisture ranges from 20 to 25 percent.

The clay was found to disperse readily and mix with water without difficulty. It was screened to remove large lumps, vegetation, and some chert particles large enough to damage the pumps.

Even though the "prescription" or bentonite grout had some special properties required in some work, there seemed to be no apparent advantage which would justify its use in preference to clay-cement grout for the types of work listed above. To substantiate this statement, a comparison of some of the properties of the two mixtures as determined in the laboratory follows. (table 2)

The clay-cement grout is insoluble, easily mixed and placed, low in cost; it has high density, high shearing strength, and rapid final set. It does not tend to separate during injection.

Chickamauga Project

The Chickamauga work was unusual but about typical for "prescription grouting" work and it will be described briefly. The Chickamauga limestone on which the dam is founded is covered by overburden 40 feet thick through which a cutoff had to be established under the earth embankments on both sides of the river. Below the 50-foot thickness of limestone which made up the upper part of the bedrock was a layer of shale and meta-bentonite which effectively protected the pure, underlying limestone from solution.

The limestone was highly cavernous due to solution along many joints. Most of the joints were steeply inclined or vertical compression joints, but tension joints were developed locally in anticlinal folds. These joints served as conduits for moving ground water. A steel sheet pile cutoff obviously would have been of no benefit. This was determined after an experimental trench had uncovered the rough surface of the rock, and it was here that the first "prescription grout" was used when it was found necessary to literally surround the test trench with a grout cutoff before the inflowing ground water permitted excavation to the rock surface.

The plan, in general, involved first grouting with "prescription grout" two rows of holes which were drilled parallel to the cutoff line, one upstream 40 feet and the other downstream 40 feet. It reached the key bed of shale about 50 feet into rock. This reduced or eliminated the inflow of water into the trench. Second, excavating the trench with a bottom width of 25 feet at bedrock through the overburden along the entire length of both embankments; removing all badly weathered rock to depths of 40 feet; cleaning out and repacking with concrete all cavities and solution channels exposed in the trench; stage grouting with clay-cement the underlying rock strata through wagon drill holes 40 feet deep, spaced 12 inches on centers; drilling large (36-inch diameter) holes for exploring, cleaning, and concreting cavities too large to be grouted successfully; constructing a low concrete core wall on top of bedrock to tie into the backfill along the center of the trench; backfilling the trench with impervious material to the original ground level. (figure 5)

A total of approximately 1,400,000 cubic feet of all kinds of grout was injected. Clay-cement predominated, but over 330,000 cubic feet of "prescription grout" was injected.

South Holston Project

South Holston and Boone projects had features which were particularly well adapted for treatment with clay-cement grout. These dams are typical in that they are tributary storage and power projects involving moderately high heads. The foundations of the various parts of these structures were unique in that it was possible to use the clay-cement grout for remedial treatment. Had it been found necessary to use neat-cement grout in these cases, the cost of foundation treatment would have been exceedingly high.

South Holston Dam is located on the South Fork of the Holston River approximately six air miles southeast of Bristol, Tennessee-Virginia. The main structure is an earth and rock fill dam 290 feet high. Its foundation is sandstone and shale, and since the treatment involved the grouting of small joints, faults and open bedding planes, neat-cement grout was used entirely.

An auxiliary saddle dike which was built on the rim 11 miles upstream from the main dam is located in Holston Valley which is developed in a faulted anticline in the Knox dolomite and the Lenoir limestone between two parallel ridges of Athens shale. The larger and higher part of the embankment was built on the southwest limb of the anticline. It is an earth fill structure and since it is founded on limestone and dolomite, its foundation treatment involved the grouting of numerous large cavities.

The axis of the dike is essentially normal to the trend of the Holston River Valley. It has a maximum height of 50 feet and is 3700 feet long. Sinkholes, both active and inactive, dot the valley floor showing the effects of solution in the underlying limestone and dolomite. The proposed grout cutoff was explored along its base line a distance of 5700 feet, but after careful study it was executed for only a distance of 5250 feet.

Exploratory drilling and geologic investigations were started in March 1942. Before suspension of work in October on orders of the War Production Board, a line of diamond drill holes on 100-foot centers, together with three 36-inch diameter calyx holes had been substantially completed along the axis of the embankment. After the war, activities were resumed and grouting operations began in June 1949.

Logs of core borings made for exploration and foundation treatment showed that solution cavities under the saddle dike were, in general, confined to moderately shallow depths, but in limited areas they extended to considerable depths. In the 300-foot wide belt of Lenoir limestone at the south end of the dike, nearly all of the cavities were within the upper 30 feet of bedrock. All of these cavities were small and restricted mainly to bedding planes. In the upper part of the Knox dolomite just north of the Lenoir outcrop, the lower limit of solution cavities was about 50 feet below the top of bedrock. However, at one place, a cavity was found along a fault which extended 110 feet into bedrock.

The most cavernous rock in the entire saddle dike area lay in a 400-foot band next to the north. The cavities had been developed along bedding planes, joints and faults to a maximum depth of 240 feet below the surface. Many of the beds of dolomite were sandy and brittle which caused them to fracture easily and made them very susceptible to deep solution.

The area on the northwest limit of the anticline repeated many of the conditions previously encountered. There were several areas of good rock near the surface, but under the north end of the dike there were also local areas of deep solution cavities on faults or joints. The deepest cavity penetrated here

was 170 feet below the surface, but the rock between the faults was not dissolved below about 70 feet.

It was decided to drill and grout in stages starting with holes for stage No. 1 on 50-foot centers. This plan made use of the previously drilled holes, which were on 100-foot centers, by drilling the new holes midway between the old ones. The depths of all exploration and grout holes were determined in the field and based on geologic data obtained from previously drilled holes. When first stage holes had been grouted, the operation was repeated in the second stage on 25-foot centers, provided the grout acceptance in the first stage had been sufficiently great to make it necessary; then third and fourth stage holes on 12.5-foot and 6.25-foot centers, respectively, were drilled and grouted as necessary. In the seriously weathered rock with deep interconnected solution channels where the fourth stage holes failed to effect an impervious grout curtain it was necessary to drill and grout fifth stage holes on 3.175-foot centers. The grout in each hole was injected in zones chosen from the logs by the geologist and controlled by packer settings.

The cutoff consumed 1,711,171 cubic feet of clay-cement grout. With the large quantities of material required, the choice of the much cheaper clay grout was prudent. Even so, it was necessary to reduce pressures, use accelerators, and control the placing of packers and hole spacing in order to keep the grout consumption as low as possible and still do the job. Also, in the rim areas which were grouted beyond and above the limits of the dike, the top of the grout curtain was brought up only to normal pool elevation. There was no grouting to provide for temporary storage in the surcharge zone.

Boone Project

Boone Dam is located on the South Fork of the Holston River just downstream from the mouth of the Watauga River. It is approximately 10 miles were of Johnson City, Tennessee, and 10 miles east of Kingsport, Tennessee. It is a concrete structure, 782 feet long, with non-overflow spillway and powerhouse sections flanked with an earth fill embankment 750 feet long on the right bank. The maximum static head on the turbines is 125 feet. Remedial treatment under the concrete structure consisted of mining clay-filled cavities, backfilling with concrete, and following with a neat-cement grout curtain. The earth embankment and switchyard were consolidated with clay-cement grout and the cutoff curtain under the earth fill and right rim was also constructed of clay-cement grout.

The strata underlying these structures are composed almost entirely of limestones with a few scattered beds of dolomite, both of which represent the Conococheague member of the Knox group of Upper Cambrian age. Limestones throughout the area are light to dark gray in color with a medium- to fine-grain, slightly sandy texture. The dark gray, banded variety is definitely in predominance. Where dolomite was encountered it was primarily light gray, medium-grained, and sandy.

Structurally, the right reservoir rim is situated in the southeast limb of an anticline which has been faulted and shoved to the northwest. This fact accounts for the extensive jointing and many small faults found throughout the area. Under most of the right reservoir rim there appears to be no major flexures, the strike and dip remaining close to N 27° E and 50° SE as they were under the main structure. However, near the point where the rim joined

the massive ridges, folding and faulting have contorted the beds to such an extent that dips range from 15 to 90°. In this area of extreme folding and faulting, brecciated and recemented zones are very common. The lack of key beds throughout the rim has made it virtually impossible to estimate the amount of displacement along various faults.

Consolidation of the original ground and the upper zone of weathered rock underlying the entire earth embankment required the drilling of 54 holes and grouting with a 12 to 1 mix of clay-cement grout. The holes were drilled in a regular pattern with 50-foot spacing, staggered in alternate rows. The injection totaled 136,721 cubic feet of grout.

Following the consolidation grouting, a continuous cutoff curtain was constructed. Holes 40 to 135 feet in depth were drilled along the upstream third of the embankment and grouted to refusal using 90,356 cubic feet.

Because of the pinnacled rock surface and dissolved nature of the underlying rock in the switchyard, the area was consolidated to strengthen the foundation. This program was carried out by drilling holes with diamond core drills 8 to 10 feet in rock and grouting to refusal with a lean mixture of clay-cement grout. Grouts containing 12 parts clay to 1 part cement were found to be satisfactory for this work. The holes were not laid out according to a regular pattern but were located where their need was apparent. Thirty-two holes were drilled and grouted; the injection totaled 86,233 cubic feet. Grout consumption varied from 13 to 8931 cubic feet per hole.

On the right rim, core borings indicated three distinct areas in which deep weathering existed, while the other areas showed relatively shallow weathering where treatment was executed on a limited scale. Drill holes varied from 54 to 298 feet in depth, and 680,386 cubic feet of grout was needed. For the entire project, 993,696 cubic feet of clay-cement grout was injected.

All drilling and grouting was done in stages starting with holes 50 feet on centers and continuing in some areas to the sixth stages where the holes were about 1.5 feet apart. Packers were used to control the grouting in zones from the bottom upward.

CONCLUSIONS

The clay-cement grout which has been used by the TVA is a material which is especially adapted to the treatment of large cavities in limestones and dolomites. It must be admitted that its use by TVA has been extensive, but it must not be assumed that because of the speed of injection and the amounts used that a great deal of careful planning is not necessary to properly use it. Clay-cement grouts, which depend primarily on local sources near the site for the larger part of the ingredients, can be used safely if, (1) such a material somewhat less durable than neat-cement grout can be used, (2) mixing and placing techniques can be devised that take advantage of local clays, and (3) the cavernous foundation will be stabilized by the material.

This material is limited to use in the correct locations. It is not a cure-all material that can be used in place of more costly, but much more durable, neat-cement grout. In the Valley of the Tennessee River its use has been justified by geologic conditions and low cost. In the injection of over 5-1/4-million cubic feet of clay-cement grout, TVA has saved, over the use of neat-cement grout, approximately $3,000,000.

CLAY-CEMENT GROUT MIXES WITH CALCIUM CHLORIDE

PROPORTIONS BY VOLUME[1]

CLAY-1 CEMENT-1 WATER-1[2]				CLAY-2 CEMENT-1 WATER-1.5[3]			
$CaCl_2$		Initial Set		$CaCl_2$		Initial Set	
Percent	Pounds[4]	Hours	Minutes	Percent	Pounds[4]	Hours	Minutes
10	9.4		40	20	18.8		10
9	8.5	1	40	15	14.0	1	25
8	7.5	2	00	12	11.2	3	11
7	6.6	2	21	10	9.4	4	10
6	5.6	2	31	9	8.5	4	25
5	4.7	3	11	8	7.5	4	50
4	3.8	4	11	7	6.6	5	50
3	2.6	5	20	6	5.6	6	10
2	1.9	6	00	5	4.7	6	30

(1) Based on clay from South Holston Saddle Dam Barrow Pits.

(2) 1 Bag Cement 1 Cubic Foot Clay 7.5 Gallons Water.

(3) 1 Bag. Cement 2 Cubic Feet Clay 13.3 Gallons Water.

(4) Pounds Per Bag of Cement in Mixer.

TABLE NO. 1

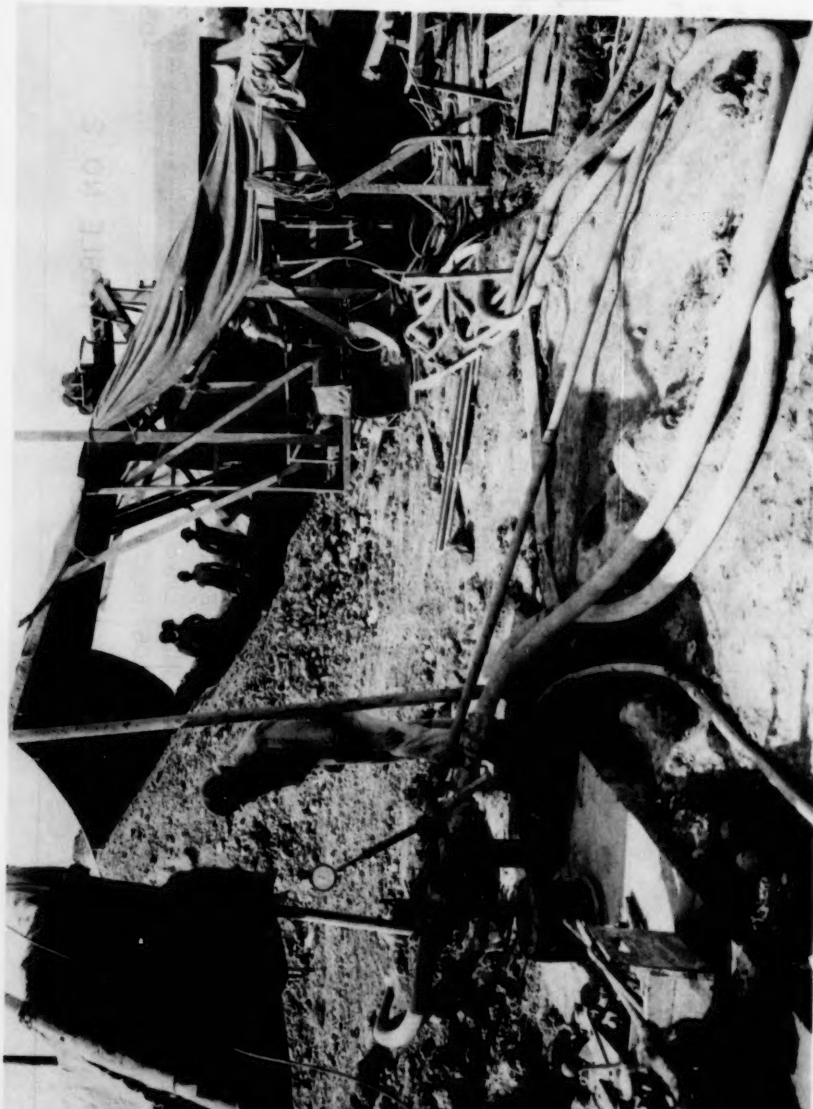
Bentonite Mix - Properties	
Water	6 Parts
Cement	$1\frac{1}{2}$ Parts
Sand	4 Parts
Bentonite	$\frac{1}{2}$ Parts
Samples Not Compressed	
Cost per cubic yard	$ 3.95
Linear srinkage - percent	1.5
Shear-pounds per square inch	13.6
Permeability coefficient-feet per year	0.55
Density-pounds per cubic foot	94.6
Initial setting time-hours	12.0
Final setting time-days	7.0
Sample Compressed 30 Pounds Per Square Inch	
Water-solid ratio	0.988
Estimated shear-pounds per square inch	22.0
Initial setting time-hours	1.0
Final setting time-hours	13.0
Linear. srinkage - percent	36.9
Height change after	+0.06

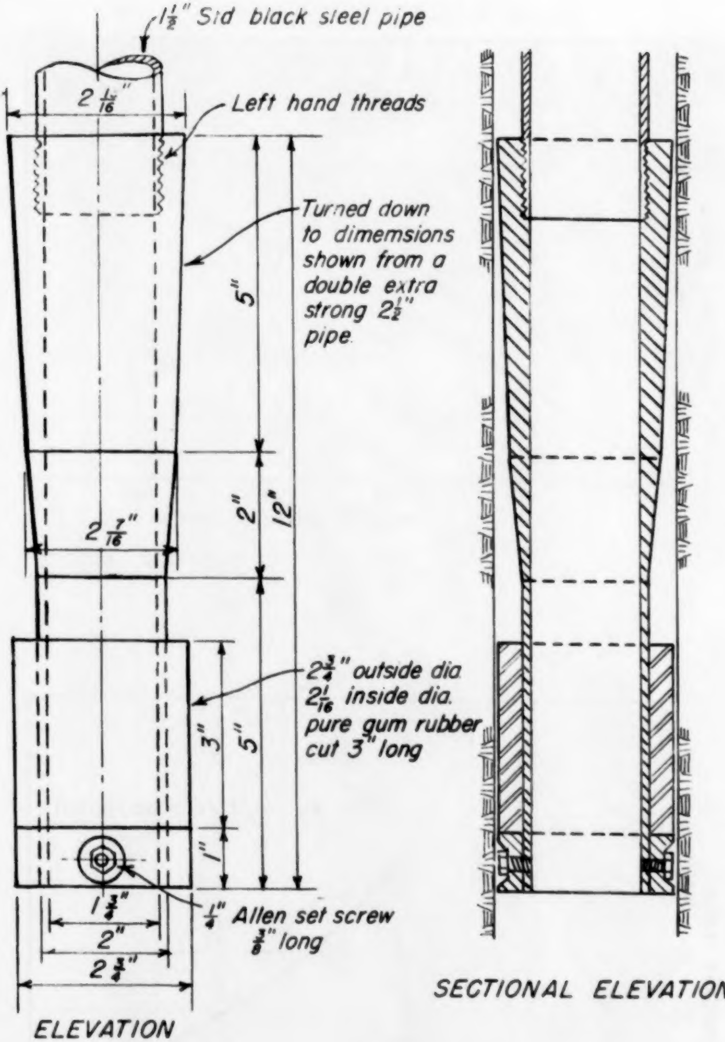
* Field test

Clay-Cement Mix - Properties	
Water	6 Parts
Cement	$1\frac{1}{2}$ Parts
Clay	8 Parts
Samples Not Compressed	
Cost per cubic yard	$ 2.92
Linear srinkage - percent	1.0*
Shear - pounds per square inch	33.9
Permeability coefficient - feet per year	9.5
Density-pounds perccubic foot	103.5
Initial setting time-hours	11.5
Final setting time-days	5.0
Sample Compressed 30 Pounds Per Square Inch	
Water-solid ratio	0.763
Estimated shear-pounds per square inch	--
Initial setting time-hours	0.5
Final setting time-hours	17.5
Linear srinkage - percent	26.7
Height change after	-0.8

TABLE NO. 2

FIGURE 1 - TYPICAL CLAY-CEMENT GROUT PLANT





NOTES:
This type packer can usually be set in water by suddenly lowering with top of 1½" pipe closed. If packer is grouted in hole it may be salvaged by means of left hand threads.

FIGURE NO. 2

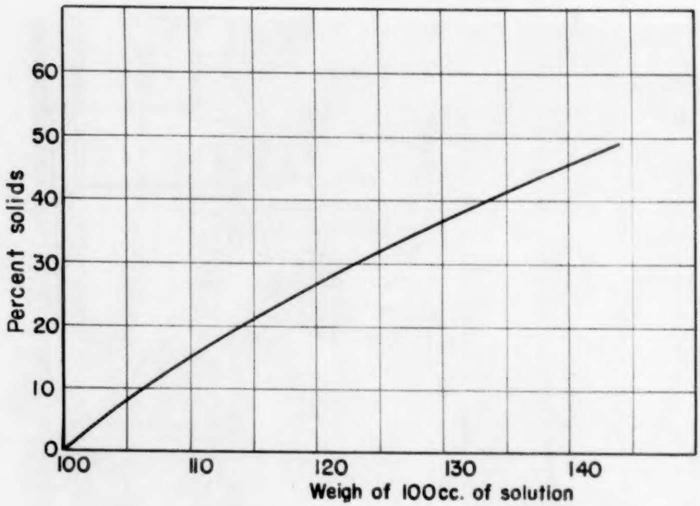


FIGURE NO. 3

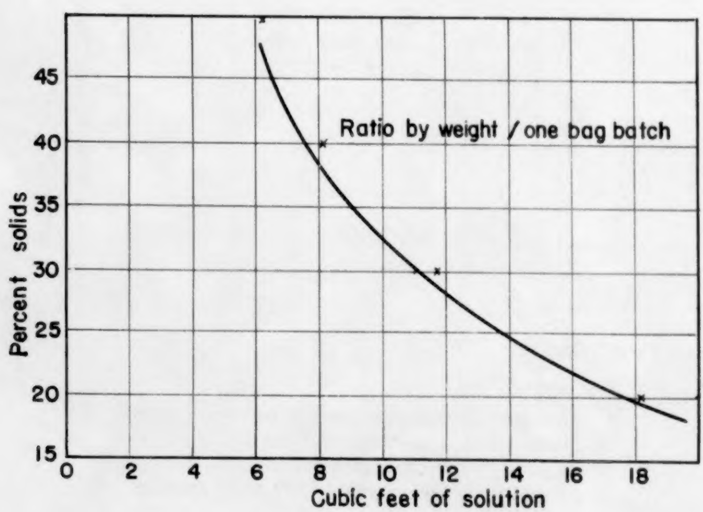


FIGURE NO. 4

FIGURE 5 - CUTOFF TRENCH AT CHICKAMAUGA DAM

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Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION
(Proc. Paper 1559)

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SEEPAGE THROUGH FOUNDATIONS CONTAINING DISCONTINUITIES[a]

Closure by Elbert E. Esmiol

ELBERT E. ESMIOL,[1] A.M. ASCE.—The discussions of J. P. Elston and H. F. Hoffman are appreciated. Both express a common criticism of geological appraisals for sites such as these; that is: the work is not sufficiently precise and detailed. The cases cited in the paper were presented in order to provide a basis for improving geological evaluations.

Mr. Elston wonders whether the discontinuities at Wickiup Dam, Enders Dam and Granby Dikes could not have been discovered during the construction period. These anomalies were not detected until the structures became operational, although competent geological advice was used. The conditions which prevented establishing these anomalies are explained in the paper. In the case of Granby Dikes, the delaying of construction to treat the foundation was considered less desirable financially than the procedure followed.

Mr. Elston believes that more extensive exploration and study in the preconstruction stage will discover most variations in the foundation. This is undoubtedly true. In the cases presented, most of the variations were discovered before placing the structures in operation; but important discontinuities were not discovered until after construction even though the best available engineering procedures were used to delineate foundation conditions.

A general recommendation for more foundation exploration than that in accepted practice may increase the cost out of all proportion to the importance of the structure without obtaining the needed information. Instead, concentrating effort on critical design problems or on problems where existing knowledge is inadequate, with special attention to minor geological details, would be a better approach.

Mr. Hoffman's questions about the composition of and the development of the joint system in the North Park Formation at Granby Dikes are as well described on pages 5 and 6 of the paper as available information permits. This information includes a petrographic description. The ages of the geological formations at Enders Dam given on page 7 of the paper are believed to be correct. The hydraulic height of McMillan Dam is 48 feet. It is based on the current practice of measuring hydraulic height from the original ground surface to the crest.

Mr. Hoffman re-emphasizes the fact that the area to be flooded should be subjected to exploration in which he includes determination of the physical, chemical, and biochemical characteristics. He further implies, by his comments on the examples, that much of this information may be obtained by

a. Proc. Paper 1143, January, 1957, by Elbert E. Esmiol.

1. Civ. Eng., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

reviewing geological literature available in libraries. Such procedures were followed in studying these cases, but the evidence shows that these were not enough. Field exploration and laboratory testing were required for design purposes.

In light of the facts for the representative cases of Wickiup Dam, Granby Dikes, Enders Dam, and McMillan Reservoir, the best available engineering and geological procedures may not disclose all of the important discontinuities in a foundation. Hence, designing for seepage assumes the status of a calculated risk. The amount of risk depends upon the accuracy with which foundation conditions have been delineated. Recognizing foundation discontinuities and evaluating their effect on seepage should improve the foundation treatment design.

RELATIVE DENSITY AND SHEAR STRENGTH OF SANDS[a]

Closure by T. H. Wu

T. H. WU,[1] A.M. ASCE.—The discussions contributed by Messrs. Seymour-Jones, Prugh, and Nishida are much appreciated. In the closure, the writer wishes to comment on some general considerations of relative density and shear strength that have received attention in the discussion.

It appears at present, that there are at least three factors that govern the relative density of cohesionless soils. They are the particle size distribution, mean particle size, and particle shape. Whether the process of sedimentation should be considered as a primary factor or not is questionable. There is as yet, too little quantitative information available to warrant any conclusions; but the writer's opinion is that it is a secondary factor that influences only the particle size distribution and the mean particle size.

One important reason for the large differences in relative density between the Mississippi Valley sand and the material studied by the writer is the particle size distribution. It should be noted that the uniformity of the Mississippi Valley sand changes but little with increasing particle size,(1) whereas, in the writer's study, the standard deviation representing the particle size distribution curve increases rapidly with increasing mean diameter. The fact that the particle size is an important factor in itself is brought out by the data on the Mississippi Valley sand. Soils with same frequency curve slopes but different peak diameters exhibit different relative densities.(1)

The relationship between relative density and consolidation pressure constitutes a somewhat different problem. The consolidation test data suggested the possibility that the large variation in relative density is a result of differences in the compressibility. It is true that the relative density of a given sand should be a function of the imposed effective stress as pointed out by Mr. Nishida. However, the erratic nature of the subsoil would certainly obliterate any relationship that might exist. Neither was it possible to detect any influence of depth on the penetration resistence. Therefore, the data were presented with no corrections for the effective overburden pressure.

The analysis of the elements that contribute to the shear strength is even less certain. The writer is in agreement with Mr. Seymour-Jones in that besides relative density, the particle size distribution and particle shape may also be important factors. In addition to the data contained in Professor Burmister's article(2) results of tests published by W. G. Holtz and H. J. Gibbs(3) demonstrate that the shear strength of gravelly soils is influenced by particle size distribution and particle shape as well as relative density. The maximum particle size, however, was found not to be an important element.

a. Proc. Paper 1161, January, 1957, by T. H. Wu.

1. Asst. Research Prof., Michigan State Univ., Lansing, Mich.

Finally, the methods of achieving the maximum and minimum densities deserve attention. At present, a number of different procedures are being used in practice. Considering the extremely large range in the size of specimens and the maximum particle size involved, standardization of the procedure would be most difficult, however desirable this may be. On the other hand, the writer believes that the fundamental relations between the relative density and the other significant physical properties are far more important than the absolute values of the relative density. Under the circumstances, the measure of relative density is still an arbitrary procedure.

The definitions of the statistical measures are given as follows:

$$\bar{x} = \frac{\sum_{i=1}^{N} w_i x_i}{\sum_{i=1}^{N} w_i} \qquad (1)$$

$$\sigma = \left[\frac{1}{N} \Sigma (x-\bar{x})^2 \right]^{1/2} \qquad (2)$$

The notations in the two equations are:

$\bar{x}$ = the mean

x_i = the variables (i = 1, 2, ----N)

w_i = the weight assigned to x_i (In the present case w_i is the % falling in a given size range)

σ = the standard deviation

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2. Burmister, D. M., The Importance and Practical Use of Relative Density in Soil Mechanics, Proc. Amer. Soc. for Testing Materials, V 48, 1948.

3. Holtz, W. G. and Gibbs, H. J., Triaxial Shear Tests on Previous Gravelly Soils, Proc. Amer. Soc. of Civil Engrs., V 82, No. 867, Jan. 1956.

A REVIEW OF THE THEORIES FOR SAND DRAINS[a]

Discussion by Yoshichika Nishida

YOSHICHIKA NISHIDA.[1]—The author has presented a good review that is very useful for performing calculations on sand drains. Mr. Richart stated that including the effects of variable void ratio did not contribute to the explanation of secondary consolidation, and this fact is surely agreed with by the writer. The second consolidation seems to be mainly due to the creep or the flow of clay, and it occupies, in some cases, more than 50% of consolidations. Mr. Richart checked the influences of well spacing, and his conclusion, that the radial flow dominated the consolidation-time behavior, is easily understood. The fact that doubling of the diameter of well influence, or the well spacing, causes an increase in the time for 90% consolidation by roughly a factor of 6 agrees with the value of 5.65 obtained from the study in Japan that the time for consolidation is proportional to the 2.5 power of the well spacing. The writer wishes to know the most suitable spacing from the practical point of view, since a too much smaller spacing brings the influence of remoulding and smearing to the clay.

a. Proc. Paper 1301, July, 1957, by F. E. Richart.

1. Instructor of Kanazawa University, Ueno-hon-machi Kanazawa-shi, Japan.

DETERMINATION OF THE 0.02 mm FRACTION IN GRANULAR SOILS[a]

Discussion by Irving Sherman

IRVING SHERMAN,[1] A.M. ASCE.—The calculation of an average relation between the fraction passing the No. 200 sieve and the fraction finer than 0.02 mm is a useful method under certain conditions for reducing the number of hydrometer analyses which are necessary. Mr. Johnson is to be commended for his approach, which he has noted is valid only for similar soils.

However, such a correlation may suffer from the lack of a causal physical relationship between the two variables. If the fraction passing the No. 200 sieve is considered to be unity, the fraction finer than 0.02 mm may be anything between unity and zero. The previous discussion by H. Y. Fang[2] shows some of the possible deviations between observed and calculated values, and other particle-size analysis data can be presented to illustrate even greater deviations.

If statistical correlations are to be used in an effort to reduce the number of hydrometer analyses, it seems preferable to base the correlations on soil properties which are physically more closely related to the soil fraction which is to be estimated. This writer previously presented[3] a description of such a method which is based on the relationship between surface area and adsorbed moisture content of soil.

The use of statistical correlations to reduce the volume of laboratory analysis will necessarily cause some reduction of accuracy in the end result. If accuracy is important, such correlations may still be useful as tools for preliminary investigation and as means for selecting specific samples for detailed analysis.

a. Proc. Paper 1309, July, 1957, by R. W. Johnson.
1. Assoc. Civ. Engr., Los Angeles County Flood Control District, Pasadena, Calif.
2. Proc. Paper 1430-35, November 1957, by H. Y. Fang.
3. Sherman, Irving, A Rapid Substitute for Textural Analysis, Journal of Sedimentary Petrology, Vol. 21, No. 3, pp. 173-177, September 1951.

THIXOTROPIC CHARACTERISTICS OF COMPACTED CLAYS[a]

Discussion by E. S. Barber

E. S. BARBER,[1] A.M. ASCE.—The authors measured thixotropy on samples molded by a kneading type of compaction while Leonards reported a constant strength with time for samples molded by static compaction.

Some thixotropy is caused by redistribution of non-uniformly distributed moisture to make a more uniform, and therefore stronger, material.

Table 2 shows no thixotropy for good mixing and static molding; but shows appreciable thixotropy due to redistribution of moisture after poor mixing and after kneading compaction which produces non-uniform moisture across shear surfaces.

The equalization of moisture after shearing is a factor in thixotropy of undisturbed clays after remolding.

a. Proc. Paper 1427, November, 1957, by H. B. Seed and C. K. Chan.
1. Civ. Engr., Arlington, Va.

Table 2 — Thixotropy of Clay Near Optimum Moisture (17) and Maximum Density (109)

Mixing	Molding	Curing	Unconfined Compressive Strength Kips per square foot
good	static	2 minutes	4.12
good	static	1 hour	4.22
good	static	1 day	4.12
poor	static	2 minutes	3.21
poor	static	1 hour	3.58
poor	static	1 day	4.22
good	kneading	2 minutes	4.12
good	kneading	1 hour	4.77
good	kneading	1 day	5.41

LATERITE SOILS AND THEIR ENGINEERING CHARACTERISTICS[a]

Discussion by Edward S. Barber

EDWARD S. BARBER,[1] A.M. ASCE.—Table 8 presents supplementary data on the physical properties of lateritic soils from Costa Rica and Panama. The tests were run by the U. S. Bureau of Public Roads in connection with the Inter-American Highway.

These soils show higher strengths and less swelling than soils from the United States with similar densities and plasticity indexes. One sample showed increased plasticity when air-dried and greater strength after disturbance than when undisturbed.

a. Proc. Paper 1428, November 1957, by K. S. Bawa.
1. Civ. Engr., Arlington, Va.

TABLE 8 - LATERITIC SOIL TEST RESULTS

Country Location	Costa Rica Alajuela	Panama Las Lomas	Panama Rio Chico	Panama East of David
Depth, feet			1.5	2.5
Per cent passing:				
No. 10 sieve		100	100	100
No. 40	100	95	98	98
No. 200	96	87	89	90
0.005 mm.	78	64	68	70
0.002 mm.			63	64
0.001 mm.	63	52	57	58
Liquid limit	59	54	62*	64
Plastic limit	26	13	34*	23
Specific gravity	2.79	2.72	2.64	2.72
Ignition loss,%			14	16
Silica, %			44	33
Alumina, %			35	38
Iron oxide,%			6	11
Silica-sesqui-oxide ratio	1.55	1.15	1.93	1.24
Kaolinite indicated by differential thermal analysis, %			70	60
<u>Standard compaction</u>				
Optimum moisture, %	41	29		
Maximum density, pcf	77	88		
CBR at optimum and 95% density	5	12		
Swell	0.5	0.1		
CBR at optimum and 90% density	3	10		
Swell	0.5	0.1		
<u>Modified compaction</u>				
Optimum moisture, %	36	24		
Maximum density, pcf	86	99		
CBR at optimum and 95% density	8	45		
Swell	2.1	0.3		
CBR at optimum and 90% density	3.5	27		
Swell	2.1	0.5		
<u>Undisturbed</u>				
Moisture content, %			25	35
Dry density, pounds per cu. ft.			87	71
California bearing ratio			9	4
Swell			0.0	0.0
Unconfined compressive strength, kips per square foot			2.3**	
Maximum stress difference for lateral stress of one kip/sq.ft.			5.4**	

* Corresponding values for soil not air-dried were 55 and 27.

** Corresponding strengths remolded at same moisture and density were 4.5 and 10.2; after remolding and curing two days they were 16.6 and 17.4.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1449 is identified as 1449 (HY 6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1957.

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OCTOBER: 1387(CP2), 1388(CP2), 1389(EM4), 1390(EM4), 1391(HY5), 1392(HY5), 1393(HY5), 1394(HY5), 1395(HY5), 1396(PO5), 1397(PO5), 1398(PO5), 1399(EM4), 1400(SA5), 1401(HY5), 1402(HY5), 1403(HY5), 1404(HY5), 1405(HY5), 1406(HY5), 1407(SA5), 1408(SA5), 1409(SA5), 1410(SA5), 1411(SA5), 1412(EM4), 1413(EM4), 1414(PO5), 1415(EM4)[c], 1416(PO5)[c], 1417 (HY5)[c], 1418(EM4), 1419(PO5), 1420(PO5), 1421(PO5), 1422(SA5)[c], 1423(SA5), 1424(EM4), 1425(CP2).

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c. Discussion of several papers, grouped by divisions.